Missouri Department of Transportation

Bridge Division

Bridge Design Manual

Section 3.71

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GENERAL

Use Load Factor design method, except for footing pressure and pile capacity where the Service Load design method shall be used.

In some cases, Service Load design method may be permitted on widening projects, see Structural Project Manager.

The terms, Intermediate Bents and Piers, are to be considered interchangeable for this Manual Section.

DESIGN UNIT STRESSES (also see Section 4 - Note A1.1)

(1) Reinforced Concrete
Class B Concrete (Substructure) fc = 1,200 psi f'c = 3,000 psi
Reinforcing Steel (Grade 60) fs = 24,000 psi fy = 60,000 psi
n = 10
$Ec = W^{1.5} \times 33 \sqrt{f'c}$ (AASHTO Article 8.7.1) (*)

(2) Reinforced Conrete (**)
Class B-1 Concrete (Substructure) fc = 1,600 psi f'c = 4,000 psi
Reinforcing Steel (Grade 60) fs = 24,000 psi fy = 60,000 psi
n = 8
$Ec = W^{1.5} \times 33 \sqrt{f'c}$ (AASHTO Article 8.7.1) (*)

(3) Structural Steel
Structural Carbon Steel (ASTM A709 Grade 36)
fs = 20,000 psi fy = 36,000 psi

(4) Pile Footings
For pile capacity, see Bridge Manual Sections 1.4 and 3.74. Also, see the Design Layout if pile capacity is indicated.

(5) Spread Footing
For foundation material capacity, see Bridge Manual Section 1.4. See the Design Layout for allowable footing pressure.

(6) Overstress
The allowable overstresses as specified in AASHTO Article 3.22 shall be used where applicable for Service Load design method.

(*) Use W = 150 pcf, $Ec = 60,625 \sqrt{f'c}$

(**) May be used for special cases, see Structural Project Manager.

LOADS

(1) Dead Loads

See Bridge Manual Section 1.2.

(2) Live Loads

As specified on the Design Layout.

Impact of 30% is to be used for the design of beam, web supporting beam and top of columns. No impact is to be used for bottom of column, tie beam or footing design.

(3) Wind and Frictional Forces

See Bridge Manual Section 1.2.

(4) Temperature and Shrinkage

The effect of normal and parallel components to the bent shall be considered. When bearings with high friction coefficients are used (see Bridge Manual Sections 1.2 and 3.31) or for long bridge lengths, the columns and footings are to be analyzed for moments normal to the bent due to the horizontal deflection of the top of the bent.

(5) Buoyancy

If specified by the Structural Project Manager, or by the Design Layout.

(6) Earth Pressure

Bents are to be analyzed for moments due to equivalent fluid pressure on columns and web where the ground line at time of construction, or potential changes in the ground line, indicate.

(7) Earthquake

The design of all bridges in Seismic Performance Categories A, B, C & D are to be designed by earthquake criteria in accordance with this bridge manual.

(8) Special Stability Situations

When indicated by the Design Layout or by the Structural Project Manager, piers must be analyzed for special loadings as directed (i.e., differential settlement).

(9) Collision

Where the possibility of collision exists from railroad traffic, the appropriate protection system, for example Collision Wall, shall be provided (See the Design Layout).

(10) Collision Walls

Collision walls are to be designed for the unequal horizontal forces from the earth pressure, if the condition exists (See the Design Layout).

The vertical force on the collision wall is the dead load weight of the wall (✳). If a bent has three or more columns, design the steel in the top of the wall for negative moment.

✳ For footing design, the eccentricity dead load moment due to an unsymmetrical collision wall shall be considered.

DISTRIBUTION OF LOADS

Design

(1) Dead Loads

Loads from stringers, girders, etc. shall be concentrated loads applied at the centerline of bearing. Loads from superstructure, such as concrete slab spans, shall be applied as uniformly distributed loads.

(2) Live Loads

Loads from stringers, girders, etc., shall be applied as concentrated loads at the intersection of centerline of stringer and centerline of bent.

(3) Wind and Frictional Forces

See Bridge Manual Section 1.2.

(4) Temperature

Apply at the top of the substructure beam.

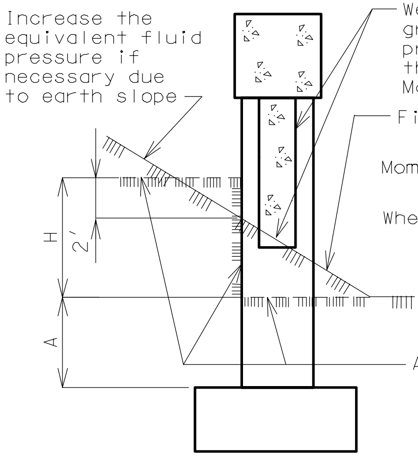
(5) Earth Loads

(a) Vertical

Vertical earth loads on tie beams shall be applied as uniform loads for a column of earth equal to 3 times the width of the beam. The weight of earth for footing design shall be that directly above the footing, excluding that occupied by the column.

The earth above the seal courses shall be considered in computing pile loads. Refer to the Design and Dimension of the Pile Footings portion of this Manual Section.

(b) Horizontal



Moment at Bottom of Column = $[(45 \text{ pcf} \times B \times H^2)/2](H/3+A)(2.0)(\ast)$

Where:

45 lbs./cu.ft. = equivalent fluid pressure

B = column diameter in area under consideration.

($\ast$) A factor of 2.0 is applied to the moment to allow for the possibility of the column resisting earth pressure caused by the earth behind the column twice the column width.

DISTRIBUTION OF LOADS (CONT.)

(6) Earthquake Loads

The design of all bridges in Seismic Performance Categories A, B, C & D are to be designed by earthquake criteria in accordance with this bridge manual.

(7) Seal Course

The weight of the seal course shall not be considered as contributing to the pile loads, except for unusual cases.

TYPES OF DESIGN

Rigid frame design is to be used for the design of Intermediate Bents and Piers.

See Manual Section 1.2 for application of loads and group loadings.

The joint between the beam and column, and web or tie beam and column, is assumed to be integral for all phases of design and must be analyzed for reinforcement requirements as a "Rigid Frame".

The joint between the column and footing is assumed to be "fixed", unless foundation fexibility needs to be considered (consult Structural Project Manager for this assessment).

If the distance from the ground line to the footing is large (*), consideration shall be given to assuming the column to be "fixed" at a point below the ground line.

* When the distance from the ground line to the top of footing is 10' or more, the unsupported height and the fixed point may be measured from the bottom of the beam to the ground line plus 1/2 of the distance from the ground line to the top of the footing.

UNSUPPORTED HEIGHT

Unsupported height is the distance from the bottom of the beam to the top of the footing.

SINGLE COLUMN

Use rigid frame design with the column considered "fixed" at the bottom for all of the conditions.

COLUMN DIAMETER CHANGE

Use a change in column diameter as required by the Design Layout or column design.

COLUMN SPACING (TRIAL)

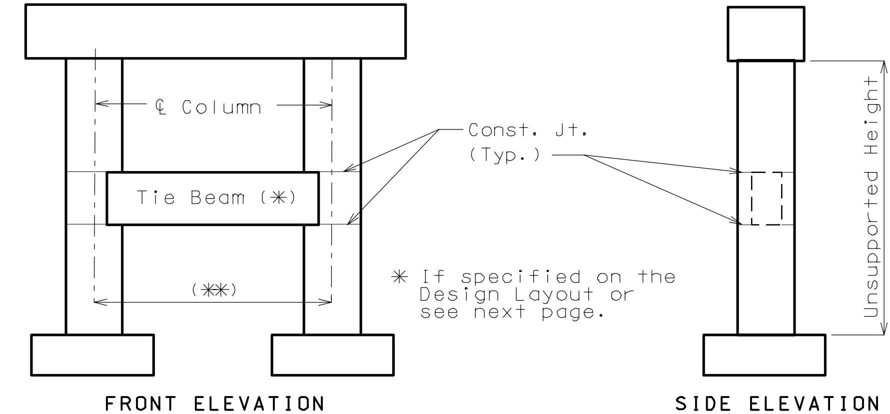
(Except Web Supporting Beam type)

Estimate centerline-centerline column spacing for a two column bent as 72% of the distance from the centerline of the outside girder to the centerline of the outside girder.

A three column bent spacing estimation is 44% of the centerline-centerline outside girder spacing.

DESIGN ASSUMPTIONS
GENERAL

Design



✳✳ For column spacings greater than 30'-0", tie beams are not to be used, unless the web supports the beam.

(1) TYPE OF DESIGN

Use rigid frame design.

(2) CHOICE OF COLUMN TYPE

Use round columns for all bridges, unless otherwise specified on the Design Layout.

(3) NUMBER OF COLUMNS

Since the spacing of the columns will depend on the number of columns used, a preliminary economic analysis should be conducted before determining the column spacing.

For this analysis, assume the following rates:

Concrete	= \$425/Cu. Yd.
Class 1 Excavation	= \$ 50/Cu. Yd.
Class 2 Excavation	= \$100/Cu. Yd.
Piles	= \$ 30/Lin. Ft.
Reinforcing bars	= Omit

Using the above rates determine the more economical number of columns.

(4) COLUMN SPACING

Column spacing (to the nearest 1") should be that which produces balanced positive and negative beam moments. A positive beam moment up to 10% larger than the negative beam moment is acceptable. For Load Factor Design, use factored moments.

Column spacing should be established with dead loads and live loads on the structure. Column spacing may be revised if other AASHTO loadings produce impractical unbalanced beam reinforcement design.

DESIGN ASSUMPTIONS (CONT.)
GENERAL (CONT.)

(5) BEAM, TOP OF COLUMN, TIE BEAM

The beams, tops of columns and tie beams shall be designed for vertical loads and maximum parallel components of horizontal forces.

Beam caps shall be designed so that service dead load moments do not exceed the cracking moment of the beam cap (AASHTO Article 8.13.3, Eq. 8-2).

Sidesway due to unsymmetrical loading shall be considered where it affects the design conditions.

(6) DESIGN OF COLUMNS

Use AASHTO Article 8.15.4. for Service Load design and AASHTO Article 8.16.4 for Load Factor design.

For bi-axial bending use the resultant of longitudinal and transverse moments.

(7) MINIMUM ECCENTRICITY FOR COLUMNS AND FOOTINGS

Apply minimum eccentricity to columns and footings.

Use the formula e(min.)= 0.6+0.03h, see AASHTO Article 8.16.5.2.8.

If minimum eccentricity controls the moment in both directions, it is necessary to use the moment in one direction only for the footing check.

(8) TIE BEAM

Use a tie beam when specified or when the unsupported height exceeds 30', except as noted.

Do not use tie beams on grade separations.

Do not use tie beams when column spacing exceeds 30'. For this situation, use a minimum column diameter of $k\ell_u/25$ ($k = 1.2$) in lieu of a tie beam.

(9) BOTTOM OF COLUMN AND FOOTINGS

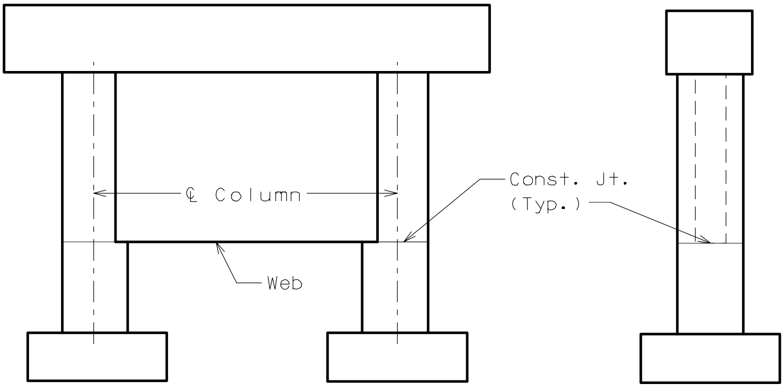
The bottom of columns and footings shall be designed for vertical loads and maximum normal and parallel components of the horizontal forces.

The footings shall be proportioned for the dimensions and for pile loads, or for bearing pressure and the position of the resultant. (See design and dimensions for footing).

(10) REINFORCEMENT

Reinforcement in beams, columns and tie beams for moments at the joints shall be based on the moments at the face of the column, beam or tie beam (equivalent square, based on areas, for round columns).

DESIGN ASSUMPTIONS (CONT.)
WEB SUPPORTING BEAM



FRONT ELEVATION SIDE ELEVATION

(1) TYPE OF DESIGN

Use rigid frame design.

(2) CHOICE OF COLUMN TYPE

Use round columns for all bridges, unless otherwise specified on the Design Layout.

(3) NUMBER OF COLUMNS

Use two or more columns, as required for more economical design.

(4) COLUMN SPACING

Space columns so that the negative moment in the beam over the outside columns requires a beam depth of 3'-0" minimum. No attempt should be made to use a column spacing (Maximum column spacing equals 35'-0") which produces equal positive and negative beam moments. The negative moment is to be determined at the face of the column (equivalent square, based on the area) for rigid frame design.

(5) BEAM AND TOP OF COLUMN

Beam (cap and web) and the top of the column shall be designed for vertical loads and maximum parallel components of horizontal forces.

Beam caps shall be designed so that service dead load moments do not exceed the cracking moment of the beam cap (AASHTO Article 8.13.3, Eq. 8-2).

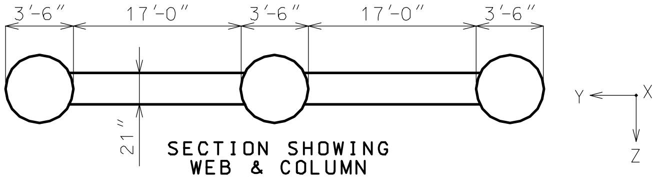
Sidesway due to unsymmetrical loading shall be considered where it effects the design conditions.

DESIGN ASSUMPTIONS (CONT.)
WEB SUPPORTING BEAM (CONT.)

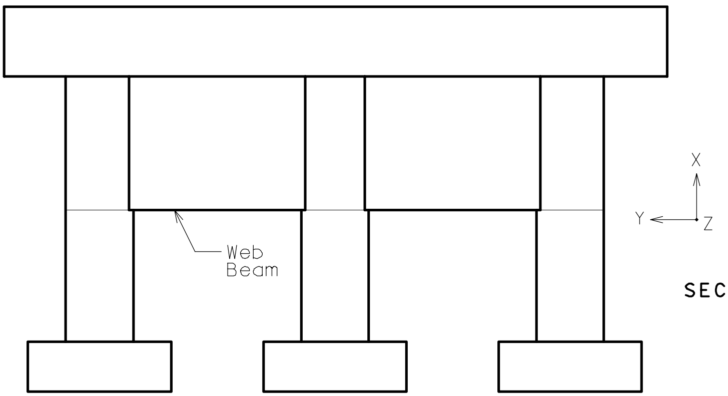
In analysis, web beams shall be modeled as plate elements. If the ability to model a web beam as a plate element is unavailable, the following simplified model may be considered:

I) The web itself is made up of several tie beams (typically 4 tie beams). The moment of inertia of individual tie beam is equal to the moment of inertia of the web in the bent's out-of-plane direction divided by the total numbers of tie beams.

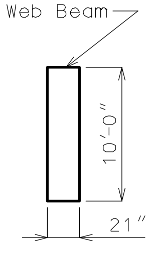
II) Any column segment which is connected to the web is treated as a prismatic member with moment of inertia in the bent's out-of-plane direction (I_z) equal to the actual column moment of inertia in that direction, and with moment of inertia in the bent's in-plane direction (I_y) equal to the total moment of inertia of web in the bent's in-plane direction divided by the total numbers of columns plus the moment of inertia of column itself. The equivalent column diameter is assumed to be $(64I_y/\pi)^{0.25}$.



SECTION SHOWING
WEB & COLUMN



FRONT ELEVATION



SECTION SHOWING
WEB

In the above example the moment of inertia of column in the bent's in-plane and out-of-plane directions can be calculated as follows:

Out-of-plane: $I_z = \pi(3.5x12)^4/64$ (in^4).

In-plane: $I_y = ((2x(17x12)x21^3)/12)/3 + \pi(3.5x12)^4/64$ (in^4).

The equivalent column diameter is then assumed to be $(64I_y/\pi)^{0.25}$. Thus column can be treated as a telescoping column and then the moment magnifier or P-δ slenderness effects can be calculated.

The web is made up of 4 tie beams. The moment of inertia of tie beam in the bent's out-of-plane direction is: $I_z = (21x(10X12)^3/12)/4$ (in^4).

DESIGN ASSUMPTIONS (CONT.)
WEB SUPPORTING BEAM (CONT.)

(6) MINIMUM ECCENTRICITY FOR COLUMNS AND FOOTINGS

Apply minimum eccentricity to columns and footings.
Use the formula e(min.) = 0.6 + 0.03h, see AASHTO Article 8.16.5.2.8.
If minimum eccentricity controls the moment in both directions, it is necessary to use the moment in one direction only for the footing check.

(7) COLUMNS AND FOOTINGS

The columns and the footings shall be designed for vertical loads and the maximum normal and parallel components of horizontal forces.

(a) Design of Columns:

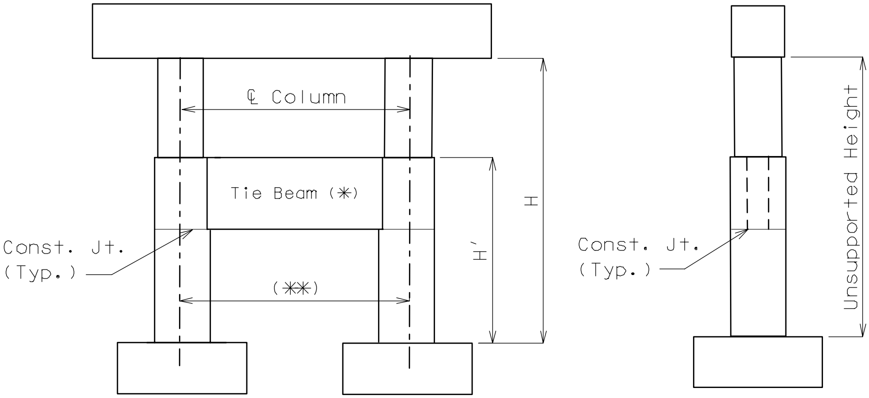
Use AASHTO Article 8.15.4 for Service Load Design and AASHTO Article 8.16.4 for Load Factor Design.

For bi-axial bending use the resultant of longitudinal and transverse moments.

(b) Footings:

The footings shall be proportioned for minimum dimensions and for the pile loads, or for the bearing pressure and the position of the resultant. (See design and dimension for footings.)

DESIGN ASSUMPTIONS (CONT.)
CHANGE IN COLUMN DIAMETER



FRONT ELEVATION SIDE ELEVATION

✳ Use tie beam if specified on the Design Layout or see next page.

✳✳ For column spacing greater than 30'-0" tie beams are not to be used, unless the web supports the beam.

(1) TYPE OF DESIGN

Use rigid frame design.

If H' ≤ 0.5H and no tie beam is used, the design may be done assuming the entire column to have the smaller diameter. This will result in a very small error.

(2) CHOICE OF COLUMN TYPE

Use round columns for all bridges, unless otherwise specified on the Design Layout.

(3) NUMBER OF COLUMNS

Use two or more columns, as required for the more economical design.

(4) COLUMN SPACING

Column spacing (to the nearest 1") should be that which produces balanced positive and negative moments. A positive beam moment up to 10% larger than the negative beam moment is acceptable. For Load Factor Design, use factored moments.

Column spacing should be established with dead loads and live loads on the structure. Column spacing may be revised if other AASHTO loadings produce impractical unbalanced beam reinforcement design.

(5) BEAM, TOP OF COLUMN, TIE BEAM

The beams, tops of columns and tie beams shall be designed for the vertical loads and the maximum parallel components of the horizontal forces.

Beam caps shall be designed so that service dead load moments do not exceed the cracking moment of the beam cap (AASHTO Article 8.13.3.Eq. 8-2).

Sidesway due to the unsymmetrical loading shall be considered where it affects the design conditions.

DESIGN ASSUMPTIONS (CONT.)
CHANGE IN COLUMN DIAMETER (CONT.)

(6) DESIGN OF COLUMNS

Use AASHTO Article 8.15.4 for Service Load Design and AASHTO Article 8.16.4 for Load Factor Design.
For bi-axial bending, use the resultant of the longitudinal and transverse moments.

(7) MINIMUM ECCENTRICITY FOR COLUMNS AND FOOTINGS

Apply minimum eccentricity to columns and footings.
Use the formula e(min.) = 0.6 + 0.03h, see AASHTO Article 8.16.5.2.8.
If minimum eccentricity controls the moment in both directions, it is necessary to use the moment in one direction only for the footing check.

(8) TIE BEAM

Use tie beam when specified or when the unsupported height exceeds 30', except as noted.

Do not use tie beams on grade separations.
Do not use tie beams when the column spacing exceeds 30'. For this situation, use a minimum column diameter of $k\ell_u/25$ (k = 1.2) in lieu of a tie beam.

(9) BOTTOM OF COLUMN AND FOOTINGS

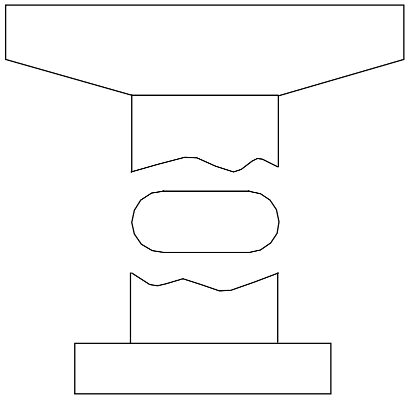
The bottom of the columns and the footings shall be designed for vertical loads and the maximum normal and parallel components of the horizontal forces.

The footings shall be proportioned for the minimum dimensions and for pile loads, or for the bearing pressure and the position of the resultant. (See design and dimensions for footings.)

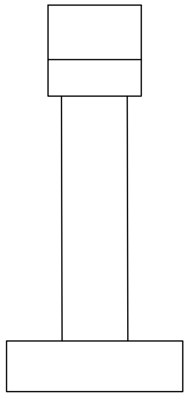
(10) REINFORCEMENT

Reinforcement in the beams, columns and tie beams for moments at the joints shall be based on the moment at the face of the column, beam or tie beam (equivalent square, based on areas, for round columns).

DESIGN ASSUMPTIONS (CONT.)
HAMMER HEAD TYPE



FRONT ELEVATION



SIDE ELEVATION

(1) TYPE OF DESIGN

Use rigid frame analysis.

(2) DESIGN OF BEAM CAP

Beam caps shall be designed so that service dead load moments do not exceed the cracking moment of the beam cap (AASHTO Article 8.13.3, Eq. 8-2).

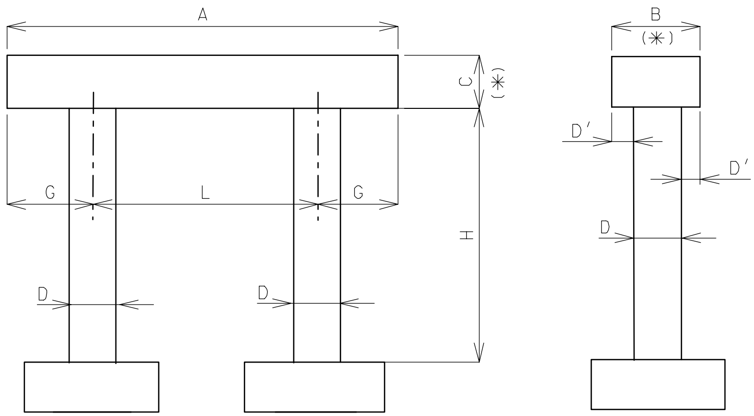
(3) DESIGN OF COLUMNS

Use AASHTO Article 8.15.4 for Service Load Design and AASHTO Article 8.16.4 for Load Factor Design.

For bi-axial bending, use the resultant of longitudinal and transverse moments.

RIGID FRAME
NO TIE (WEB) BEAM

Dimensions



FRONT ELEVATION **SIDE ELEVATION**

(1) Beam

"A" = Length to be determined by the superstructure requirements or the minimum support length required for earthquake criteria, to the nearest 1". Use square ends.

"B" = Width to be determined by the superstructure requirements, minimum support length required for earthquake criteria, or (column diameter + 6") minimum, 6" increments(✳).

"C" = Depth as required by design, 2′-6" minimum and no less than column diameter, 3" increments (✳).

(✳) Ratio of beam width/beam depth, B/C, shall be ≤ 1.25.

(2) Columns

"D" = Column diameter, 2′-6" minimum, 6" increments. (Use 3′-0" columns when beam depth exceeds 3′-6").

"D′" = Beam width overhang:
CASE I - Beam width controlled by superstructure requirements:
Minimum D′ = 3", Maximum D′ = 6"

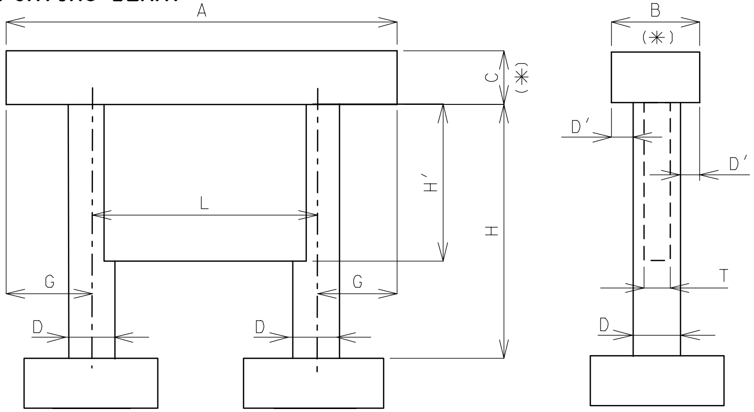
CASE II - Beam width controled by minimum support length required for earthquake criteria:
Minimum D′ = 3", Maximum D′ = 15"

"L" = Spacing as determined by design with no limit, 1" increments.
"G" = Overhang as determined by design with no limits.
"H" = Column height as required by grade and footing elevations. Use construction joint in column when H exceeds 35′-0".

NOTE: Try to keep columns and beams the same size where possible for Seismic Performance Cataegories B, C and D for economy of construction and better earthquake force distribution.

WEB BEAM
(WEB SUPPORTING BEAM)

Dimension



FRONT ELEVATION **SIDE ELEVATION**

(1) Beam

"A" = Length to be determined by the superstructure requirements or the minimum support length required for earthquake criteria, to the nearest 1". Use square ends.

"B" = Width to be determined by the superstructure requirements, minimum support length required for earthquake criteria, or (column diameter + 6") minimum, 6" increments(✳).

"C" = Depth as required by design, 3'-0" minimum and no less than column diameter, 3" increments (✳).

(✳) Ratio of beam width/beam depth, B/C, shall be ≤ 1.25.

(2) Columns

"D" = Column diameter, 3'-0" minimum, 6" increments.

"D'" = Beam width overhang:
CASE I - Beam width controlled by superstructure requirements:
Minimum D' = 3", Maximum D' = 6"
CASE II - Beam width controled by minimum support length required for earthquake criteria:
Minimum D' = 3", Maximum D' = 15"

"L" = Spacing as determined by design with a 35'-0" maximum, 1" increments.
"G" = Overhang as determined by design with no limits.
"H" = Column height as required by grade and footing elevations.

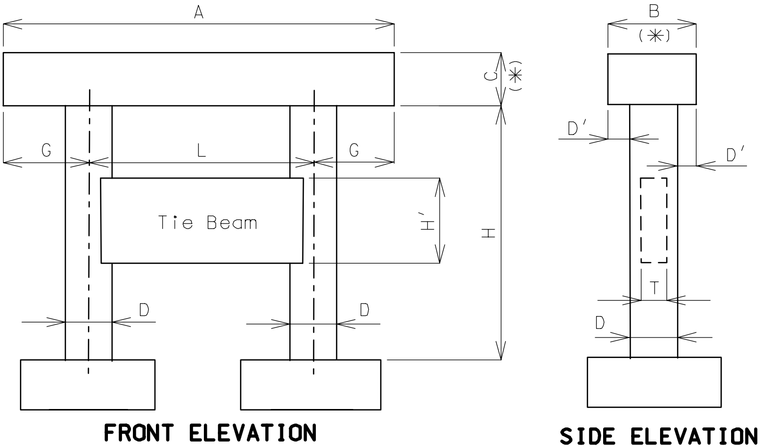
(3) Webs

"T" = Web thickness, for a 3'-0" column, use T = column diameter; for column diameter 3'-6" and over, use T = 0.5(column diameter).
"H'" = See bottom elevations of web given on the Design Layout.

NOTE: Try to keep columns and beams the same size where possible for Seismic Performance Cataegories B, C and D for economy of construction and better earthquake force distribution.

TIE BEAM

Dimensions



(1) Beam

"A" = Length to be determined by the superstructure requirements or the minimum support length required for earthquake criteria, to the nearest 1". Use square ends.

"B" = Width to be determined by the superstructure requirements, minimum support length required for earthquake criteria, or (column diameter + 6") minimum, 6" increments(✳).

"C" = Depth as required by design, 3'-0" minimum and no less than column diameter, 3" increments (✳).

(✳) Ratio of beam width/beam depth, B/C, shall be ≤ 1.25.

(2) Columns

"D" = Column diameter, 3'-0" minimum, 6" increments.

"D'" = Beam width overhang:

CASE I - Beam width controlled by superstructure requirements:
Minimum D' = 3", Maximum D' = 6"

CASE II - Beam width controled by minimum support length required for earthquake criteria:
Minimum D' = 3", Maximum D' = 15"

"L" = Spacing as determined by design with a 30'-0" maximum, 1" increments.

"G" = Overhang as determined by design with no limits.

"H" = Column height as required by grade and footing elevations.

(3) Tie Beam

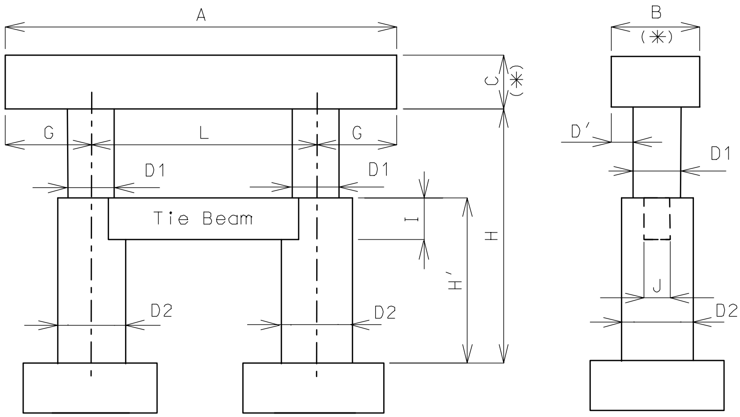
"T" = Tie Beam Thickness, minimum T = 0.5(column diameter).

"H'" = See bottom elevations of tie beam given on the Design Layout.
Minimum H' = 2T(round to the next higher foot).

NOTE: Try to keep columns and beams the same size where possible for Seismic Performance Cataegories B, C and D for economy of construction and better earthquake force distribution.

TIE BEAM WITH CHANGE IN COLUMN DIAMETER

Dimensions



FRONT ELEVATION

SIDE ELEVATION

(1) Beam

"A" = Length to be determined by the superstructure requirements or the minimum support length required for earthquake criteria, to the nearest 1". Use square ends.

"B" = Width to be determined by the superstructure requirements, minimum support length required for earthquake criteria, or (column diameter + 6") minimum, 6" increments(✳).

"C" = Depth as required by design, 3'-0" minimum and no less than column diameter, 3" increments (✳).

(✳) Ratio of beam width/beam depth, B/C, shall be ≤ 1.25.

(2) Columns

"D1" = Column diameter, 3'-0" minimum, 6" increments.

"D2" = Column diameter, (D1 + 6") minimum, see Structural Project Manager for piers or clumns in stream channels.

"D'" = Beam width overhang:

CASE I - Beam width controlled by superstructure requirements:
Minimum D'= 3", Maximum D'= 6"

CASE II - Beam width controled by minimum support length required for earthquake criteria:
Minimum D'= 3", Maximum D'= 15"

"L" = Spacing as determined by design with a 30'-0" maximum with tie beams and no limit without tie beams.

"G" = Overhang as determined by design with no limit.

"H" = Column height as required by grade and footing elevations.

"H'" = Approximately 0.5H, top of tie beam should be at the same elevaton as the top of larger diameter columns in order to minimize the number of construction joints. Top of tie beam may be located on the Design Layout.

(3) Tie Beam

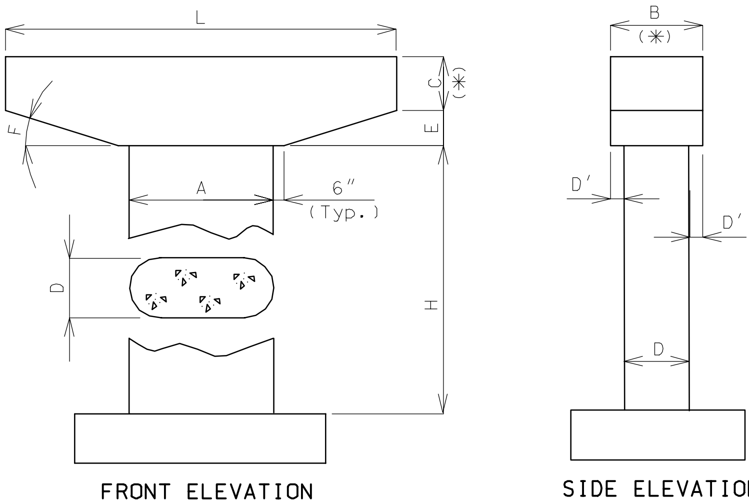
"I" = Depth as required by design, with a minmum of 3'-0", 3" increments.

"J" = Width as required by design, with a minimum of 1/2 of the column diameter D1.

NOTE: Try to keep columns and beams the same size where possible for Seismic Performance Cataegories B, C and D for economy of construction and better earthquake force distribution.

HAMMER HEAD TYPE

Dimensions



(1) Beam

"A" = Length to be determined by the superstructure requirements or the minimum support length required for earthquake criteria, to the nearest 1". Use square ends for beam.

"B" = Width to be determined by the superstructure requirements, minimum support length required for earthquake criteria, or (column width + 6") minimum, 6" increments(✳).

"C" = Depth as required by design, 2'-6" minimum and no less than column width, 3" increments (✳).

"E" = Depth as required by design (see "F")

"F" = Angle as required by design (20 degrees maximum)

(✳) Ratio of beam width/beam depth, B/C, shall be ≤ 1.25.

(2) Column

"A" = Length as required by design, approximately L/3. Use round ends for column.

"D" = Width as required by design with a minimum of 2'-6", 6" increments.

"H" = Height as required by grade and footing elevations.

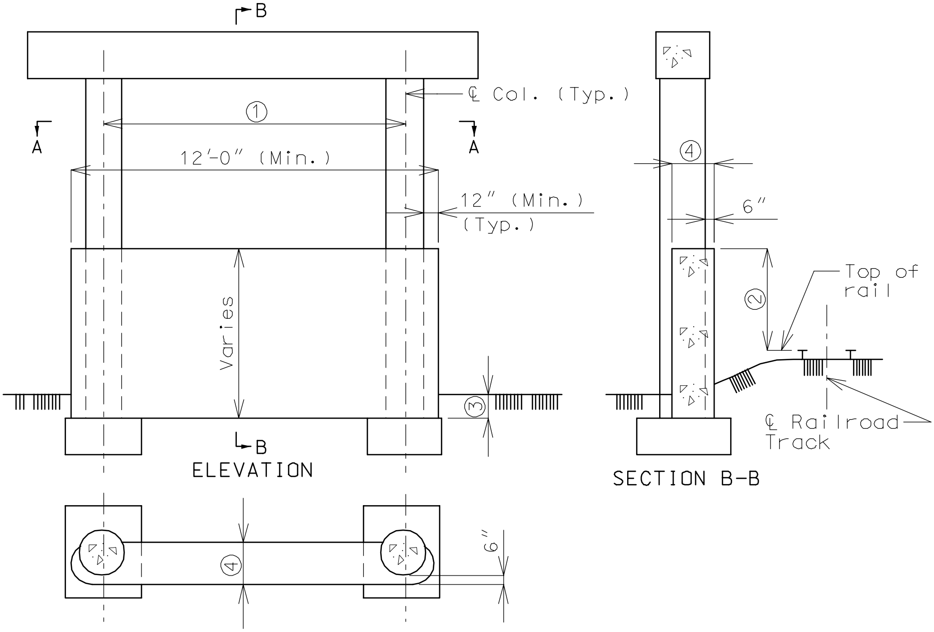
"D'" = Beam width overhang:

Case I - Beam width controlled by superstructure requirements:
Minimum D' = 3"
Maximum D' = 6"

Case II - Beam width controlled by minimum support length required for earthquake criteria:
Minimum D' = 3"
Maximum D' = 15"

Note: Try to keep columns and beams the same where possible for seismic performance catagories B, C, and D for economy of construction and better earthquake force distribution.

COLLISION WALLS
STRUCTURE OVER RAILROAD (TRACK ON ONE SIDE OF BENT)



These details are typical for bents with two or more columns.

Column faces located within 25'-0" of the centerline of track shall meet standards specified in part 2 of chapter 8 of the AREA Manual (*), except as modified in this manual. Certain railroads have specific requirements that also must be complied with. Check the Preliminary Design Layout data.

① For column spacing over 25'-0", see Structural Project Manager.

② 6'-0" Minimum for columns from 12'-0" to 25'-0" clear from the centerline of the track; 12'-0" Minimum for columns less than 12'-0" clear from the centerline of the track.

③ In general, the collision wall shall extend to at least 4'-0" below the lowest surrounding grade. For spread footing on rock the collision wall may extend to less than 4'-0" below the lowest surrounding grade with railroad's concurrence. Top of footing elevations should correspond with bottom of collision wall.

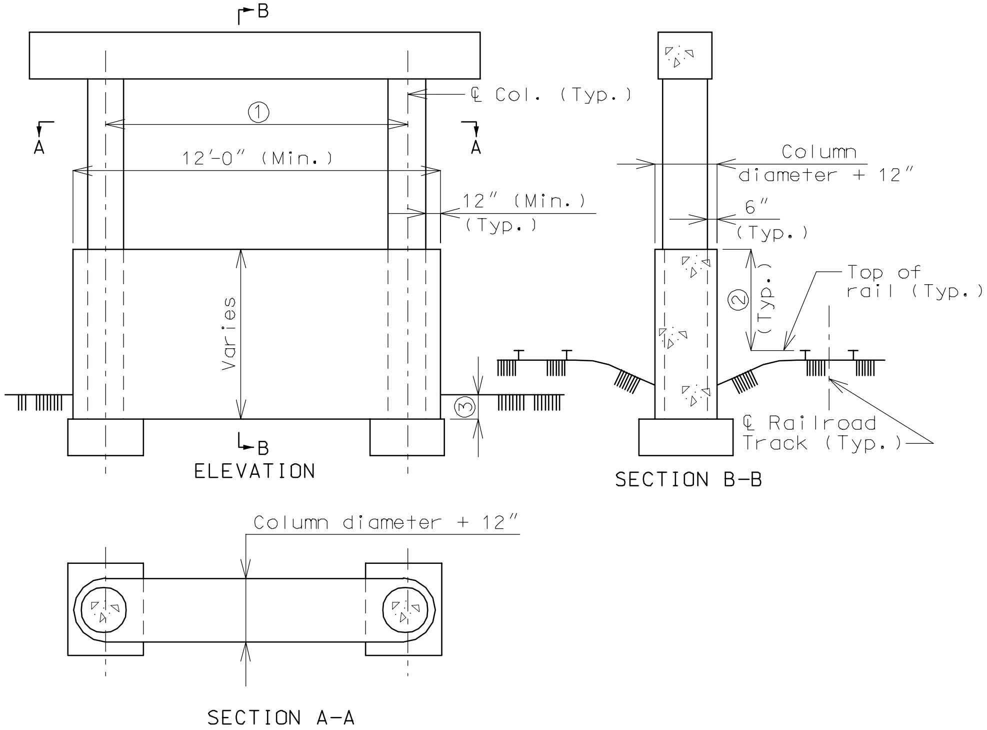
④

Diameter of Column	2'-6"	3'-0"	3'-6" and over
Width of Collision Wall	3'-0"(**)	3'-6"(**)	2'-6" (Min.)

* AREA: American Railway Engineering Association

** To facilitate construction, match the back face of collision wall to the face of column.

COLLISION WALLS
STRUCTURE OVER RAILROAD (TRACK ON BOTH SIDES OF BENT)



These details are typical for bents with two or more columns.

Column faces located within 25'-0" of the centerline of track shall meet standards specified in part 2 of chapter 8 of the AREA Manual (✻), except as modified in this manual. Certain railroads have specific requirements that also must be complied with. Check the Preliminary Design Layout data.

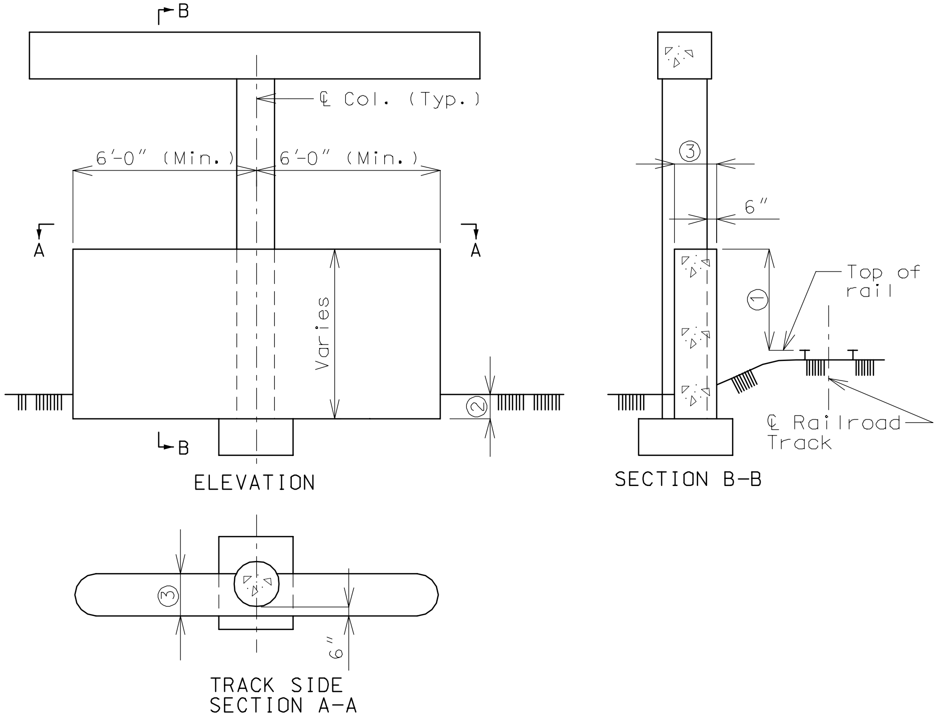
① For column spacing over 25'-0", see Structural Project Manager.

② 6'-0" Minimum for columns from 12'-0" to 25'-0" clear from the centerline of the track; 12'-0" Minimum for columns less than 12'-0" clear from the centerline of the track.

③ In general, the collision wall shall extend to at least 4'-0" below the lowest surrounding grade. For spread footing on rock the collision wall may extend to less than 4'-0" below the lowest surrounding grade with railroad's concurrence. Top of footing elevations should correspond with bottom of collision wall.

✻ AREA: American Railway Engineering Association

COLLISION WALLS
STRUCTURE OVER RAILROAD (TRACK ON ONE SIDE OF SINGLE COLUMN BENT)



These details are typical for bents with single column.

Column faces located within 25'-0" of the centerline of track shall meet standards specified in part 2 of chapter 8 of the AREA Manual (✳), except as modified in this manual. Certain railroads have specific requirements that also must be complied with. Check the Preliminary Design Layout data.

① 6'-0" Minimum for column from 12'-0" to 25'-0" clear from the centerline of the track; 12'-0" Minimum for column less than 12'-0" clear from the centerline of the track.

② In general, the collision wall shall extend to at least 4'-0" below the lowest surrounding grade. For spread footings on rock the collision wall may extend to less than 4'-0" below the lowest surrounding grade with railroad's concurrence. Top of footing elevations should correspond with bottom of collision wall.

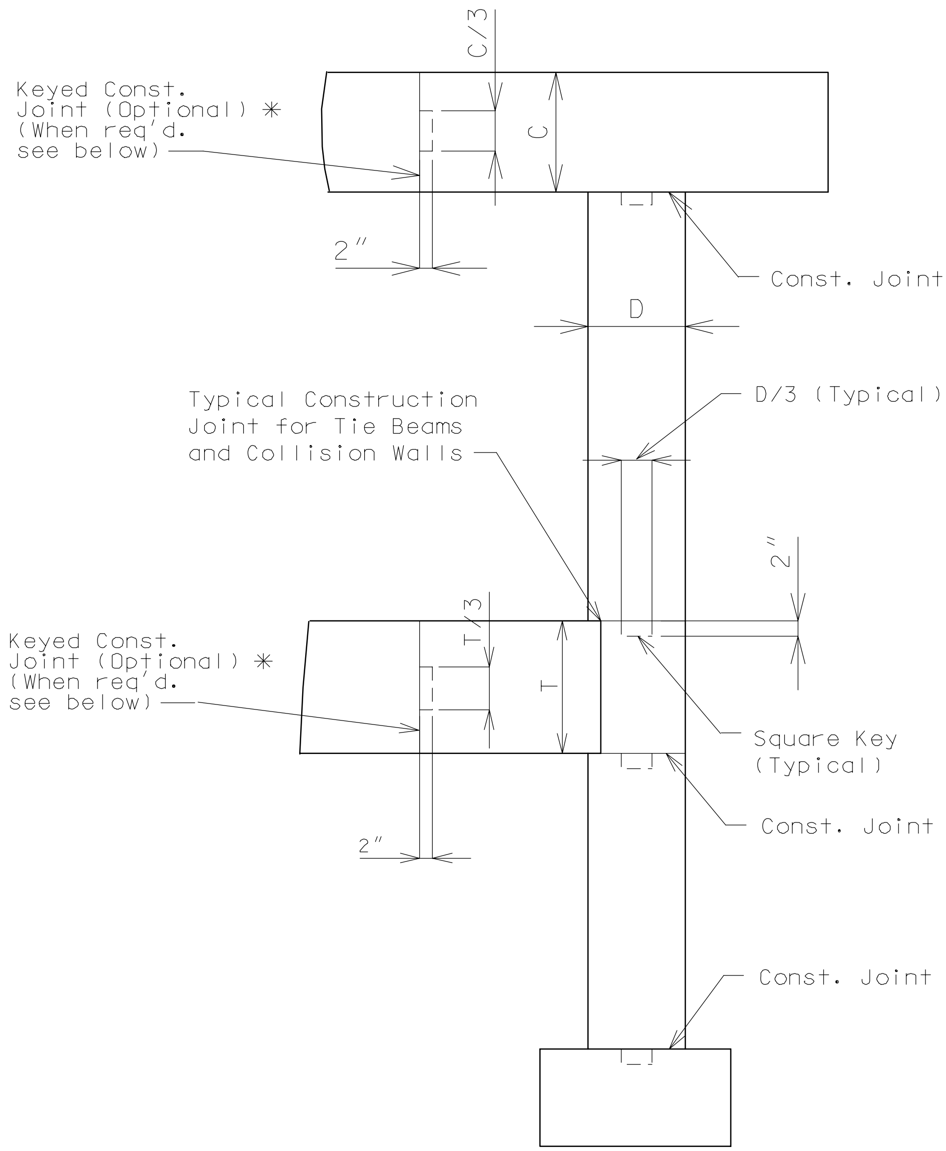
③

Diameter of Column	2'-6"	3'-0"	3'-6" and over
Width of Collision Wall	3'-0"(✳✳)	3'-6"(✳✳)	2'-6" (Min.)

✳ AREA: American Railway Engineering Association

✳✳ To facilitate construction, match the back face of collision wall to the face of column.

CONSTRUCTION JOINTS AND KEYS



PART ELEVATION

✳ Optional Construction Joints in bearing beam and tie beams:
When the total length of bearing beam exceeds 60'-0", show a keyed construction joint at or near a 1/4 point between columns in the bearing beam and tie beam. Unless required by design or stage construction, this construction joint shall be shown as optional on the plans and may be eliminated at the contractor's discretion.

For column height greater than 35'-0" with no tie beam or collision wall, place the construction joint at approximately the mid-point of the column height.

GENERAL(CONT.)
COLUMN (CONT.)
SEISMIC PERFORMANCE CATEGORIES B, C, & D

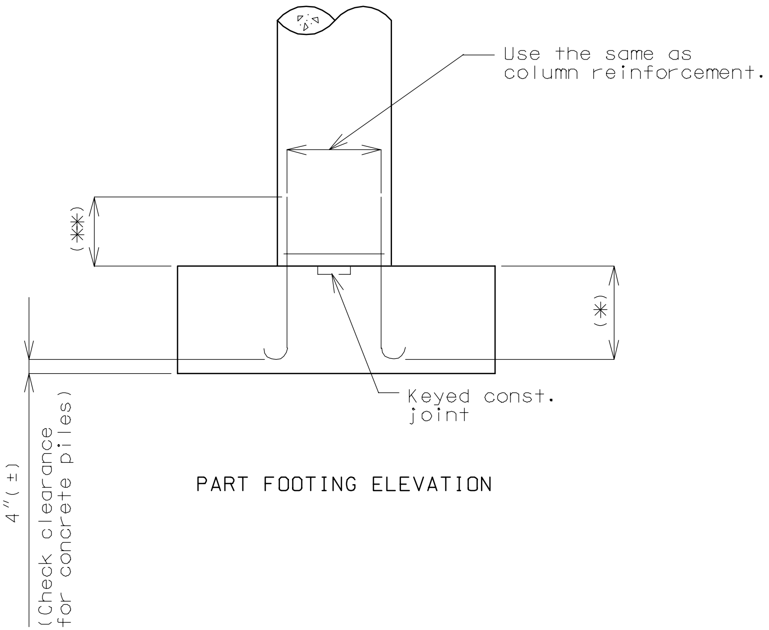
See page 3.1.7-6 of Section 6.1 Seismic Design.

GENERAL(CONT.)
COLUMN (CONT.)
SEISMIC PERFORMANCE CATEGORIES B, C, & D(CONT.)
STIRRUP BAR DETAILS

Reinforcement

See page 3.1.7-7 of Section 6.1 Seismic Design.

GENERAL(CONT.) **Reinforcement**
FOOTING
SEISMIC PERFORMANCE CATEGORY A



PART FOOTING ELEVATION

Notes:
See this Manual Section for footing reinforcement not shown.
✱ Check development length charts (Section 2.4) for when to hook column steel.
✱✱ See lap splice class C, Bridge Manual Section 2.4.

GENERAL(CONT.)

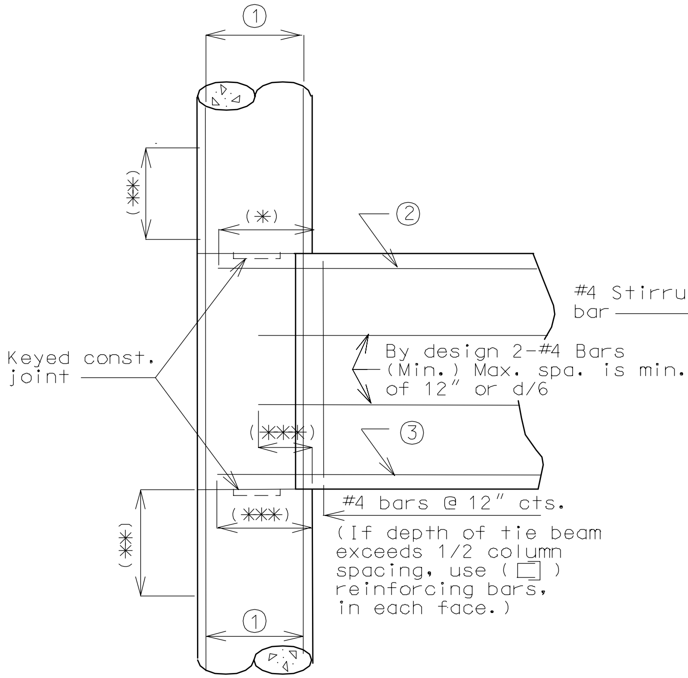
FOOTING (CONT.)

SEISMIC PERFORMANCE CATEGORIES B, C, & D

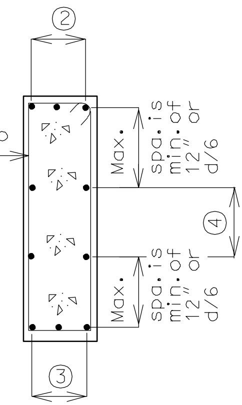
See page 3.1.7-8 of Section 6.1 Seismic Design.

GENERAL (CONT.) **Reinforcement**

TIE BEAM -SEISMIC PERFORMANCE CATERGORIES B, C & D



PART ELEVATION



TIE BEAM SECTION

(1) See column reinforcement this manual section.

(2) By design, Min. 3-#8 bars (Hook ( ) if required for bond)

(3) By design, Min. 3-#8 bars (Hook ( ) if required for bond)

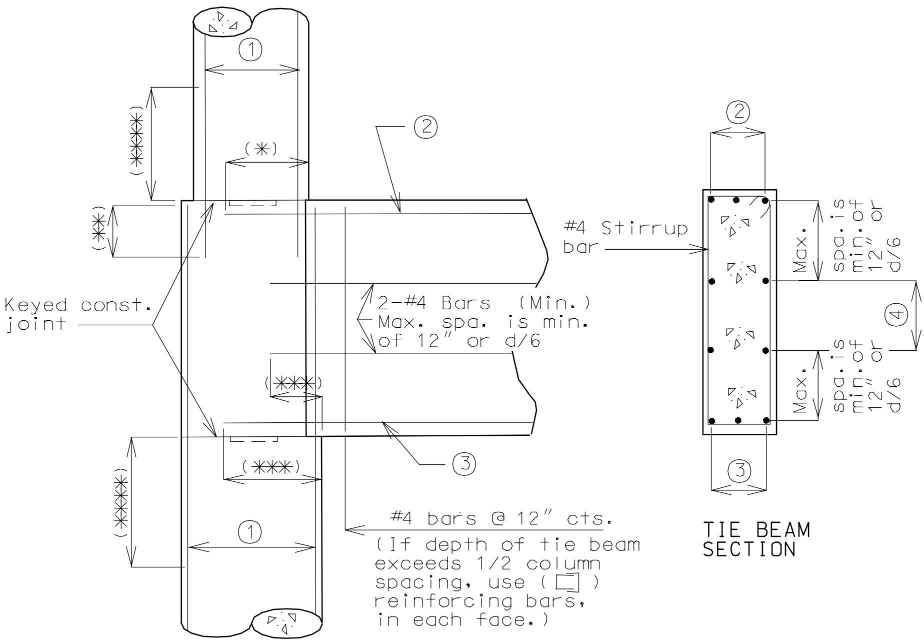
(4) Skin reinforcement by design, Min. #4 bars @ Max 12" cts.

* See development length (Top bar) in Bridge Manual Section 2.4.

** 1/4 Column height (Lap splices of vertical column reinforcement are not permitted within this length).

*** See development length (Other than top bars) in Bridge Manual Section 2.4.

TIE BEAM - SEISMIC PERFORMANCE CATERGORIES B, C, & D (CONT.)



PART ELEVATION (TIE BEAM WITH CHANGE IN COLUMN DIAMETER)

1. See column reinforcement this manual section.
2. By design, Min. 3-#8 bars (Hook ( ) if required for bond)
3. By design, Min. 3-#8 bars (Hook ( ) if required for bond)
4. Skin reinforcement by design, Min. #4 bars @ Max 12" cts.

* See development length (top bars) in Bridge Manual Section 2.4.

** For seismic performance categories B, C & D: use greater of class C lap splices in Bridge Manual Section 2.4 or 60 bar diameters.

*** See development length (other than top bars) in Bridge Manual Section 2.4

**** 1/4 Column height (Lap splices of vertical column reinforcement are not permitted within this length).

COLUMN REINFORCEMENT (CONT.)

BENDING AND DIRECT STRESS - DESIGN

(1) Lateral Reinforcement

See this Bridge Manual Section to check slenderness effects in columns and the moment magnifier method of column design.

(i) Seismic Performance Category A

Columns shall be reinforced and analyzed as "Tied Columns" - AASHTO Article 8.18.2.3, unless excessive reinforcement is required, in which case spirals shall be used. (AASHTO Article 8.18.2.2.)

(i) Seismic Performance Categories B, C & D

Spirals are required when the bridge is located in SPC B, C & D.

(2) Stress
Use AASHTO Article 8.15.4 for Service Load Design and AASHTO Article 8.16.4 for Load Factor Design.

(3) Charts
See Bridge Manual Section 1.5 (Ultimate Strength Concrete Design).

(4) Bi-Axial Bending
Use the resultant of longitudinal and transverse moments.

(5) Computer Program
(Ultimate Strength Design - BR138)
This program may also be used for the design of columns under Service Load Design by using a load factor equal to 2.86 and PH1 equal to 1.0 (AASHTO Article 8.15.4.)

COLUMN REINFORCEMENT (CONT.)
Reinforcement

SLENDERNESS EFFECTS IN COLUMNS

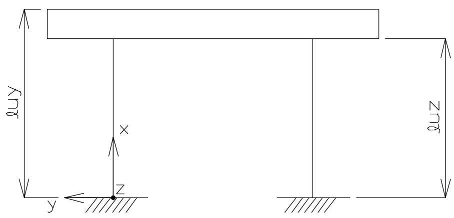
Slenderness effects should be included in the column design if the unsupported length, ℓu, is greater than 22r/k in which r = radius of gyration of column cross section.

The slenderness effects shall be considered by using either the rigorous P-δ analysis or the Moment Magnifier Method given in AASHTO Article 8.16.5.2. with consideration of bracing and non-bracing effects.

Note:
Use of the moment magnifier method is limited to members with $k\ell u/r \leq 100$, or diameter of column $\geq (k\ell u/25)$.

When a compression member is subjected to bending in both principal directions, the effects of slenderness should be considered in each direction independently.

Instead of calculating two moment magnifier, δ_b and δ_s, and performing two analyses for M_{2b} and M_{2s} in accordance with AASHTO 8.16.5.2, the following conservative, simplified moment magnification method in which only a moment magnifier due to side way, δ_s, analysis is required:



Column element coordinate system = (x, y & z)

Figure 1. Typical Intermediate Bent

a.) Column moment parallel to the bent in-plane direction:

$$M_{cy} = \delta_{sy} M_{2y} \quad (1)$$

Where

M_{cy} = Magnified Column moment in the y direction for design.
M_{2y} = Value of larger column moment in the y direction due to AASHTO group loadings.

$$\delta_{sy} = \frac{Cm}{1 - \frac{Pu}{\varnothing Pc}} \geq 1.0; \; Cm = 1.0 \quad (2)$$

= moment magnification factor for sidesway;
Pu = applied axial design load at a column;

Note: Under Service Load Design the term Pu shall be 2.5 times the axial load with $\varnothing = 1.0$.

$$Pc = \frac{\pi^2 EIy}{(k\ell u)^2} \quad (3)$$

= Euler buckling load

EI can be calculated in accordance with AASHTO 8.16.5.2 with consideration of dead load effect (i.e. β_d);

ℓu = Top of footing to top of beam cap;

k = 1.2 minimum. Use k = 2.0 for column design in longitudinal bridge direction with non-integral intermediate bents. Use k = 1.2 for column design in longitudinal bridge direction with integral intermediate bents.

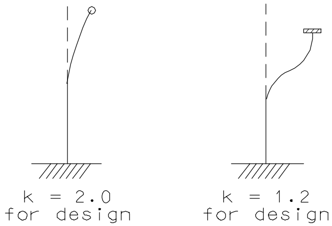


Figure 2. Boundary Conditions for Column

b.) Column moment normal to the bent in-plane direction:

$$M_{ez} = \delta_{sz} M_{2z} \qquad (4)$$

Where

M_{ez} = Magnified Column moment in the z direction for design.
M_{2z} = Value of larger column moment in the z direction due to AASHTO group loadings.

$$\delta_{sy} = \frac{Cm}{1 - \frac{\Sigma Pu}{\varnothing \Sigma Pc}} \geq 1.0;\ Cm = 1.0 \qquad (5)$$

= moment magnification factor for sidesway.

ΣPu = Summation of individual column axial design loads for a specific group loading. (i.e. Group 1 max. axial)

ΣPc = Summation of the individual column Euler buckling loads.

$$= \Sigma \frac{\pi^2 EIz}{(k\ell u)^2} \qquad (6)$$

ℓu = Top of footing to bottom beam or tie beam and/or top of tie beam to bottom of beam

k = 1.2 for prismatic column.

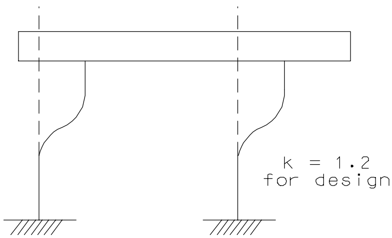


Figure 3. Transverse Bent Movement

COLUMN REINFORCEMENT (CONT.)

Reinforcement

For telescoping columns, the equivalent slenderness factor, k, and equivalent moment of inertia, I, can be estimated as follows:

a.) $I = \frac{\Sigma \ell_n I_n}{L}$ (7)

Where ℓ_n and I_n are the length ad moment of inertia of column segment n as shown in Figure 3. L is the total length of telescoping column.

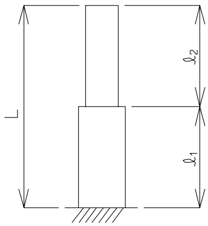
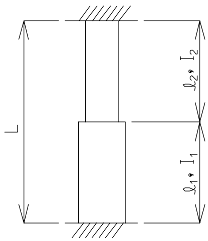


Figure 4. Telescoping Columns

b.) Equivalent slenderness factor k:

I.) Elastic buckling load, Pc, for fixed-fixed condition:



From Eq. (8), find Pc:

$$(a_1 + a_2)[d_1 + d_2 - Pc(\frac{1}{\ell_1} + \frac{1}{\ell_2})] - [c_1 - c_2]^2 = 0 \quad (8)$$

Where

$$a_1 = \frac{4EI_1}{\ell_1}; \quad a_2 = \frac{4EI_2}{\ell_2}$$

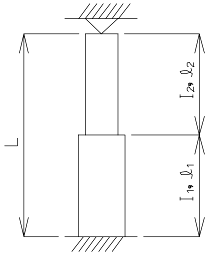
$$c_1 = \frac{6EI_1}{\ell_1^2}; \quad c_2 = \frac{6EI_2}{\ell_2^2}$$

$$d_1 = \frac{12EI_1}{\ell_1^3}; \quad d_2 = \frac{12EI_2}{\ell_2^3}$$

From Eqs. (3), (7) and (8):

$$k = \sqrt{\frac{\pi^2 EI}{PcL^2}} \quad (9)$$

II.) Elastic buckling load, Pc, for hinge-fixed condition:



From Eq. (10), find Pc:

$$(a_1+a_2)[d_1+d_2-Pc(\frac{1}{\ell_1}+\frac{1}{\ell_2})](a_2)+2b_2c_2(c_2-c_1)$$
$$-(b_2)^2[d_1+d_2-Pc(\frac{1}{\ell_1}+\frac{1}{\ell_2})]-(a_2)[c_2-c_1]^2-(c_2)^2(a_1+a_2)=0 \quad (10)$$

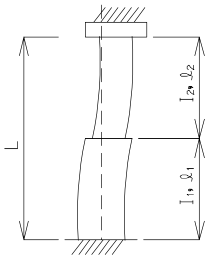
Where

$b_1 = \frac{2EI_1}{\ell_1}$; $b_2 = \frac{2EI_2}{\ell_2}$

and a_1, a_2, c_1, c_2, d_1 and d_2 are defined in Eq. (8)

Substitute Pc from Eq. (10) into Eq. (9) to obtain k.

III.) Elastic buckling load, Pc, for fixed-fixed with lateral movement condition:



From Eq. (11), find Pc:

$$[(d_1+d_2)-\frac{(c_2-c_1)^2}{a_1+a_2}-Pc(\frac{1}{\ell_1}+\frac{1}{\ell_2})][d_2-\frac{c_2^2}{a_1+a_2}-Pc(\frac{1}{\ell_2})]$$
$$-[-d_2+\frac{c_2(c_2-c_1)}{a_1+a_2}+Pc(\frac{1}{\ell_2})]^2=0 \quad (11)$$

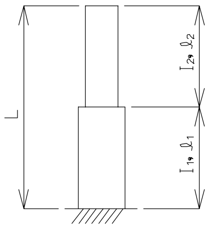
Where

a_1, a_2, c_1, c_2, d_1 and d_2 are defined in Eq. (8)

Substitute Pc from Eq. (11) into Eq. (9) to obtain k.

COLUMN REINFORCEMENT (CONT.) Reinforcement

IV.) Elastic buckling load, Pc, for fixed-free with lateral movement condition:



From Eq. (12), find Pc:

$$[d_1 + d_2 - Pc(\frac{1}{\ell_1} + \frac{1}{\ell_2}) - \frac{A_1}{\beta}][d_2 - \frac{Pc}{\ell_2} - \frac{A_3}{\beta}] - [-d_2 + \frac{Pc}{\ell_2} - \frac{A_2}{\beta}]^2 = 0 \qquad (12)$$

in which

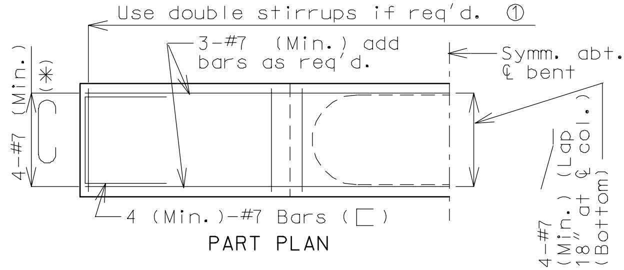
$\beta = (a_1 + a_2)(a_2) - (b_2)^2$
$A_1 = (c_1 - c_2)[a_2(c_1 - c_2) + b_2 c_2] + c_2[b_2(c_1 - c_2) + c_2(a_1 + a_2)]$
$A_2 = (c_1 - c_2)[c_2 a_2 - b_2 c_2] + c_2[b_2 c_2 - c_2(a_1 + a_2)]$
$A_3 = (c_2)[c_2 a_2 - 2b_2 c_2 + c_2(a_1 + a_2)]$

Where

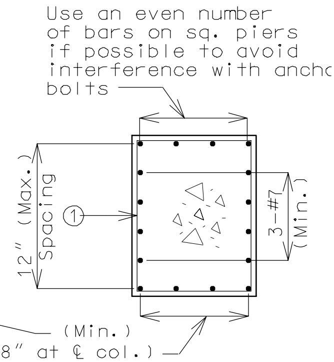
a_1, a_2, b_1, b_2, c_1, c_2, d_1 and d_2 are defined in Eqs. (8) and (10)

Substitute Pc from Eq. (12) into Eq. (9) to obtain k

HAMMER HEAD TYPE **Reinforcement**

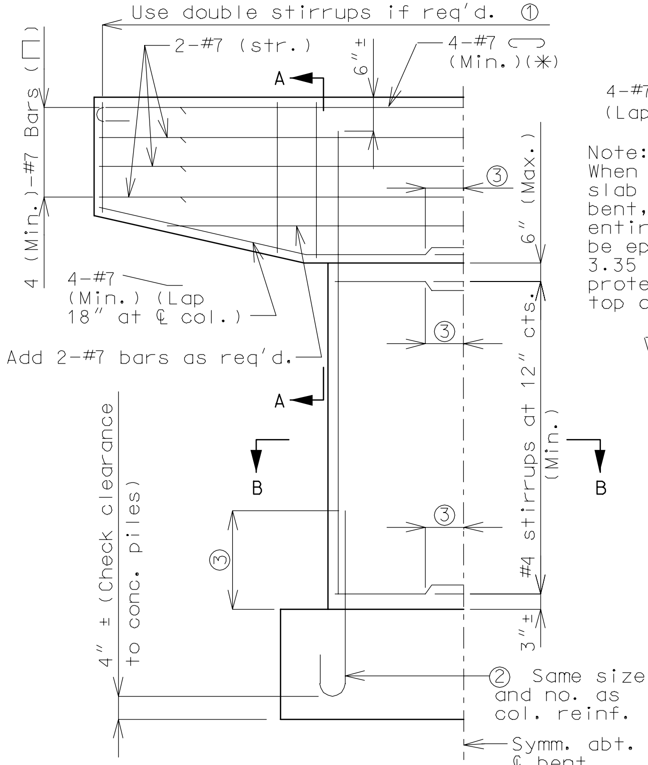


PART PLAN

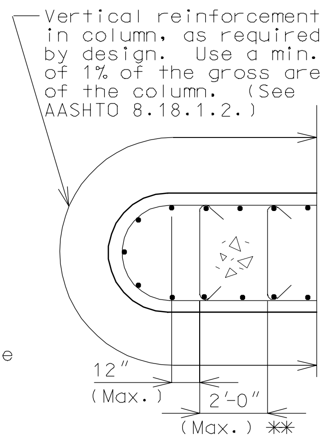


SECTION A-A

Note:
When an expansion device in the slab is used at an intermediate bent, all reinforcement located entirely within the beam shall be epoxy coated. See section 3.35 page 5.4 for details of protective coating and sloping top of beam to drain.



PART ELEVATION



PART SECTION B-B

✳ Add hooked reinforcement as required by design.

✳✳ See AASHTO Article 8.18.2.3.4 for tie requirements.

① All stirrups in beam to be the same size bar. (Use a min. spacing of 5" (6" for double stirrups), minimum stirrups are #4 at 12" cts., and maximum stirrups are #6 at 6" cts.)
Locate #4 bars (⊓) under bearings if required.
(Not required for P/S Double-Tee Girders.)

② See development length (Other than top bars) or standard hooks in tension, Ldh - Manual Section 2.4.

③ See lap splice class C - Manual Section 2.4.

GENERAL

Number, size and spacing of piling shall be determined by computing the pile loads and applying the proper allowable overstresses.

Cases of Loading:
(AASHTO Article 3.22)

Group I and Group II maximum vertical loads (refer to distribution of loads, this Bridge Manual Section).

Group III thru VI wind and/or temperature moments with applicable vertical loads.

See Sections 6 and 7 of 1996 AASHTO Division I-A for earthquake loads combined with applicable vertical loads. (✳)

Internal stresses including the position of the shear line shall then be computed.

Long narrow footings are not desirable and care should be taken to avoid the use of an extremely long footing 6′-0″ wide when a shorter footing 8′-3″ or 9′-0″ wide could be used.

Footings are to be designed for the greater of the minimum moment requirements at the bottom of the column, or the moments at the bottom of the footing.

When using the load factor design method for footings, design the number of piles needed based on the working stress design method.

(✳) The design of all bridges in Seismic Performance Categories B, C & D are to be designed by earthquake criteria in accordance with this Bridge Manual.

PILE LOADS

$P = N/n \pm M/S$
- P - Pile Loads
- N - Vertical Loads
- n - Number of Piles
- M - Overturning Moment
  If minimum eccentricity controls the moment in both directions, It is necessary to use the moment in one direction (direction with less section modulus of pile group) only for the footing check.
- S - Section Modulus of Pile Group

(A) AASHTO Group I thru VI Loads as applicable

Maximum P = Pile Capacity
Minimum P = 0 (zero)

Tension on a pile will not be allowed for any combination of forces.

Pile design force shall be calculated with consideration of AASHTO percentage overstress factors.

(B) Earthquake Loads

Point Bearing Pile

Maximum P = Pile capacity x 2 (✳✳)
(i.e., for HP 10 X 42 piles, Max. P = 56 X 2 = 112 tons/pile)
Minimum P = Allowable uplift force specified for piles in this Bridge Manual Section under Seal Course Design.

(✳✳) Two "2" is our normal factor of safety. Under earthquake loadings only the point bearing pile and rock capacities are their ultimate capacities.

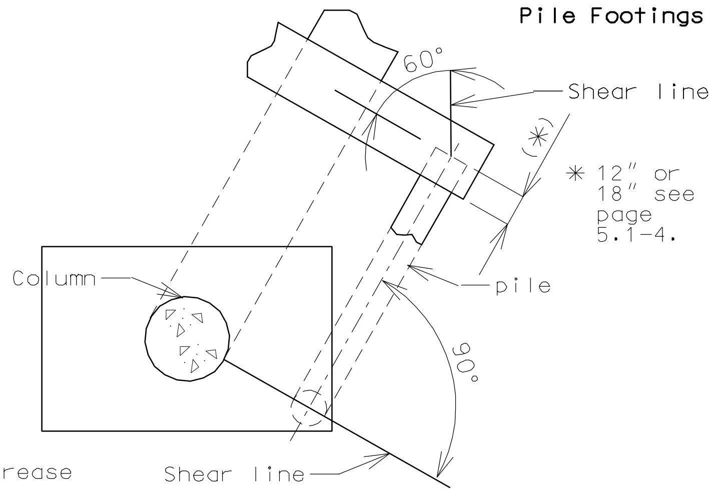
Friction Piles

Maximum P = Pile Capacity
Minimum P = Allowable uplift force specified for piles in this Bridge Manual Section under Seal Course Design.
See Bridge Manual Section 3.74 for combined axial & bending stresses in Cast-In-Place friction piles in liquefaction areas.

DESIGN & DIMENSIONS(CONT.)

Pile Footings

INTERNAL STRESSES



(1) Shear Line

If the shear line is within the column projected, the footing may be considered satisfactory for all conditions and standard #6 hairpin bars shall be used.

If the shear line is outside of the column projected, the footing must be analyzed and reinforced for bending and checked for shear stress (see following sheet, this Bridge Manual Section).

Footing depths may be increased, in lieu of reinforcement, if an increase would be more economical. (6'-0" Maximum depth, with 3" increments.)

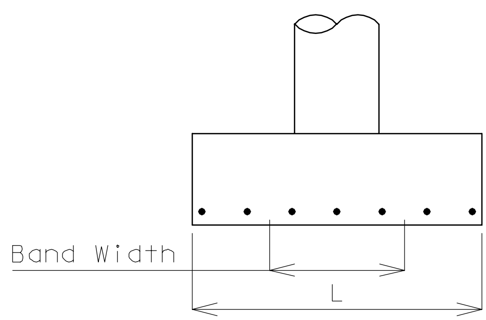
(2) Bending

The critical section for bending shall be taken at the face of the columns (concentric square of equivalent area for round columns).

The reinforcement shall be as indicated for reinforced footings, except that the standard #6 hairpin bars may be used for small footings if they provide sufficient steel area.

(3) Distribution of Reinforcement

Reinforcement in Bottom of Footing



L = Footing Length
B = Footing Width

Reinforcement shall be distributed uniformly across the entire width of footing in the long direction. In the short direction, the portion of the total reinforcement given by AASHTO Equation 4.4.11.2.2-1 shall be distributed uniformly over a band width equal to the length of the short side of the footing, B.

Band Width Reinforcement = 2(total reinforcement in short direction)/(β+1)

where β = the ratio of footing length to width = L/B

The remainder of the reinforcement required in the short direction shall be distributed uniformly outside the center band width of footing.

Reinforcement in Top of Footing

Reinforcement in the top of the footing shall be provided based on a seismic analysis for Seismic Performance Categories B, C and D. This reinforcement shall be at least the equivalent area as the bottom steel in both directions. The top steel shall be placed uniformly outside the column.

DESIGN & DIMENSIONS(CONT.) Pile Footings
INTERNAL STRESSES (CONT.)

(4) Shear
(AASHTO Article 8.15.5 or 8.16.6)

The shear capacity of footing in the vicinity of concentrated loads shall be governed by the more severe of the following two conditions.

(i)Beam shear

Critical Section at "d" distance from face of column.

b = Footing width

Service Load

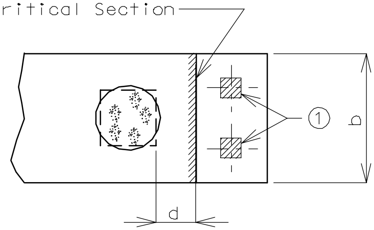
$v = V/(b\ d)$

$v_c = 0.95\sqrt{f'c}$

Load Factor

$v_u = V_u/(\text{Ø}\ b\ d)$

$v_c = 2.0\sqrt{f'c}$



PART PLAN OF FOOTING

(ii)Peripheral Shear

Critical Section at "d/2" distance from face of column.

$b_o = 4(d + \text{Equiv. square column width})$

Service Load

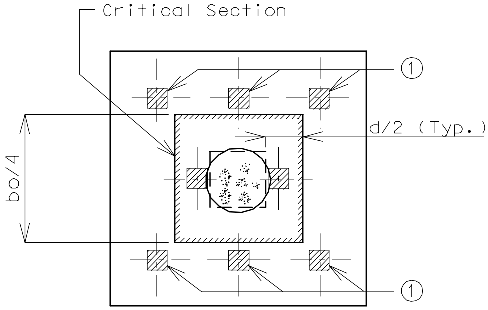
$v = V/(b_o\ d)$

$v = 1.8\sqrt{f'c}$

Load Factor

$v = V_u/(\text{Ø}\ b_o\ d)$

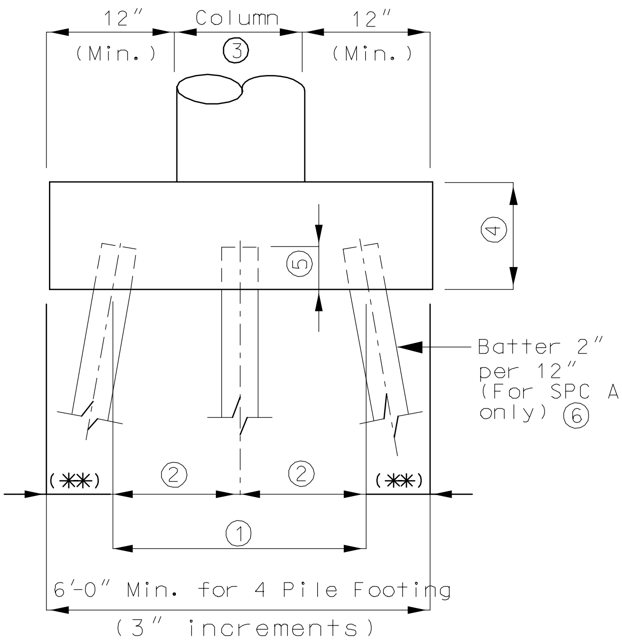
$v_c = 4.0\sqrt{f'c}$



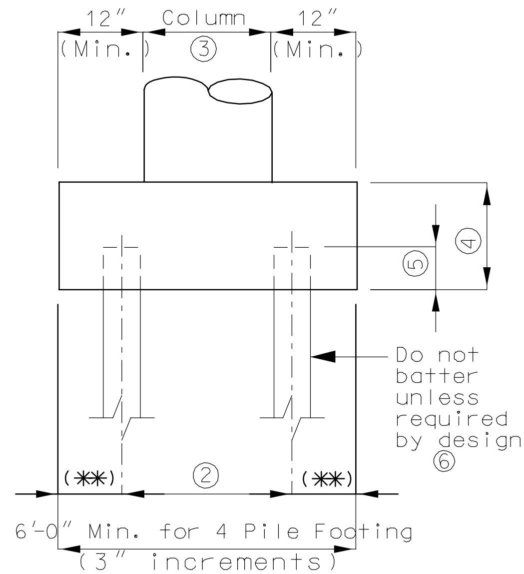
PLAN OF FOOTING

If shear stress is excessive, increase footing depth.

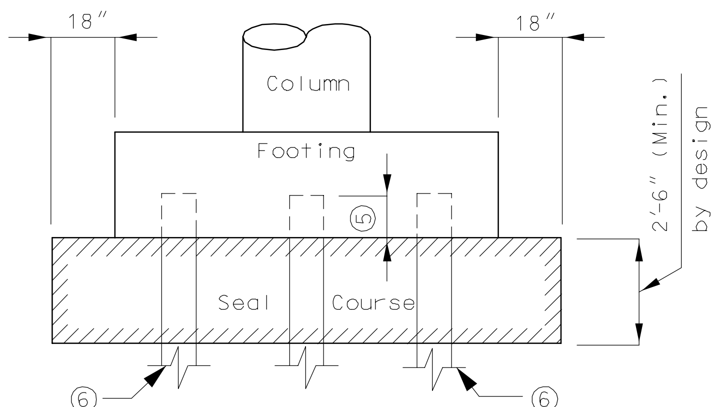
1 Piles to be considered for shear. (Center of piles are at or outside the critical section.)



SIDE ELEVATION



FRONT ELEVATION



SEAL COURSE

① Min. = 1/8 x (Distance from top of beam to bottom of footing.)

② 3′-0″ (Min.) & 6′-0″ (Max.) for steel HP piles, 14″ CIP piles. AASHTO Article 4.5.6.4 shall be considered if piles are situated in cohesive soils.

3D (Min.) and 6D (Max.) for 20″ and 24″ CIP piles. (D = pile diameter)

③ Indicates column diameter, or column length or width on a hammer head pier.

④ Min. = 2′-6″ or column diameter (✱) (Or width) for friction piles for SPC A.

Min. = 3′-0″ or column diameter (✱) (Or width) for friction piles for SPC B,C,& D.

Min. = 3′-0″ or column diameter (✱) (Or width) for steel piles for SPC A.

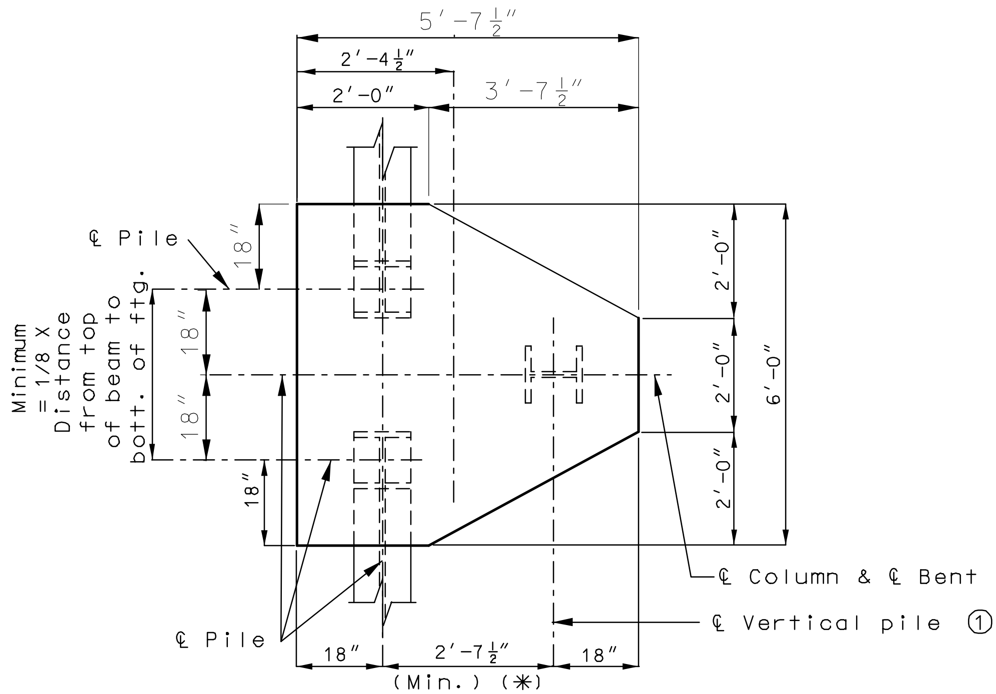
Min. = 3′-6″ or column diameter (✱) (Or width) for steel piles for SPC B, C & D.

⑤ 12″ for seismic performance category A and 18″ for SPC B, C, & D.

⑥ Use vertical piles when seal course is used.
If horizontal thrust requires pile batter - consult the Structural Project Manager.

(✱) For column diameters 4′-0″ and greater use a 4′-0″ min. footing thickness.

(✱✱) Use 18″ for steel HP piles, 14″ CIP piles, precast and prestress piles.
Use 21″ for 20″ and 24″ CIP Piles.



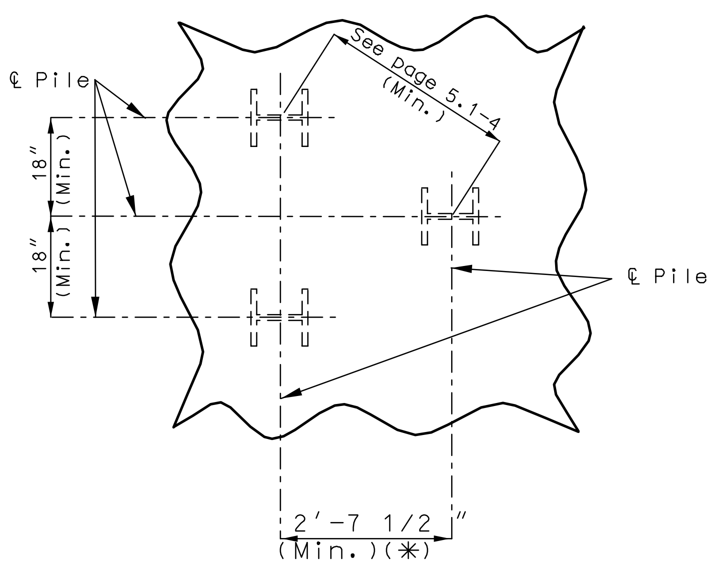
TYPICAL PLAN OF 3 PILE FOOTINGS

(minimum pile spacings)

NOTES:

Use 3 - piles on exterior footings only.

Use only HP 10x42 or friction piles on three pile footings.



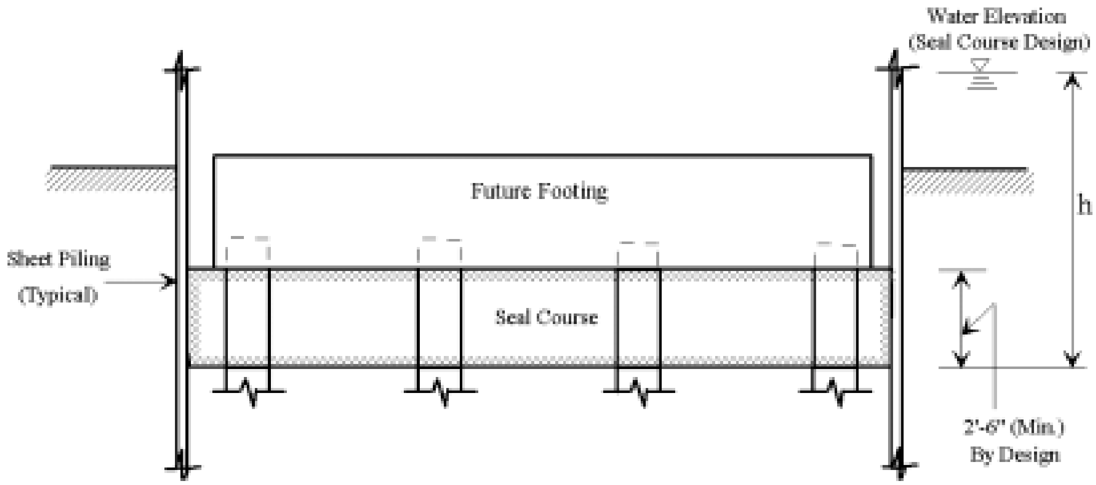
TYPICAL PLAN (STAGGERED PILE)

(7 Pile footings shall not be used.)

① If horizontal thrust requires pile batter - consult the Structural Project Manager.

(✳) The Maximum pile spacing is 4'-0".

Seal Course 5.2



NOTE: All Pile Shall be Vertical.

General Requirements:

Water elevation is to be determined for the site conditions by the preliminary design section; generally, less than the average of high and low water.

Determine the uplift force per pile by deducting the weight of the seal course and friction between seal course and sheet piling of cofferdam from the uplift force produced by the hydrostatic head "h".

Use a friction value between the seal course and the sheet piling of 2 lbs./in^2 acting on (perimeter x depth) of seal course.

Pile Pullout Force

Allowable uplift force per pile shall be determined by the minimum of:

AASHTO Table 4.5.6.2A

(1) The allowable friction capacity of pile = ultimate friction capacity of pile divided by a safety factor of 3.5. Use "SPILE" program to calculate ultimate friction capacity of pile.

AASHTO 8.16.6.2

(2) Ultimate pull-out capacity of pile due to shear failure (with maximum shear stress of $2\sqrt{f'c}$) divided by a safety factor of 3. *See Figures 1, 2 and the following example.*

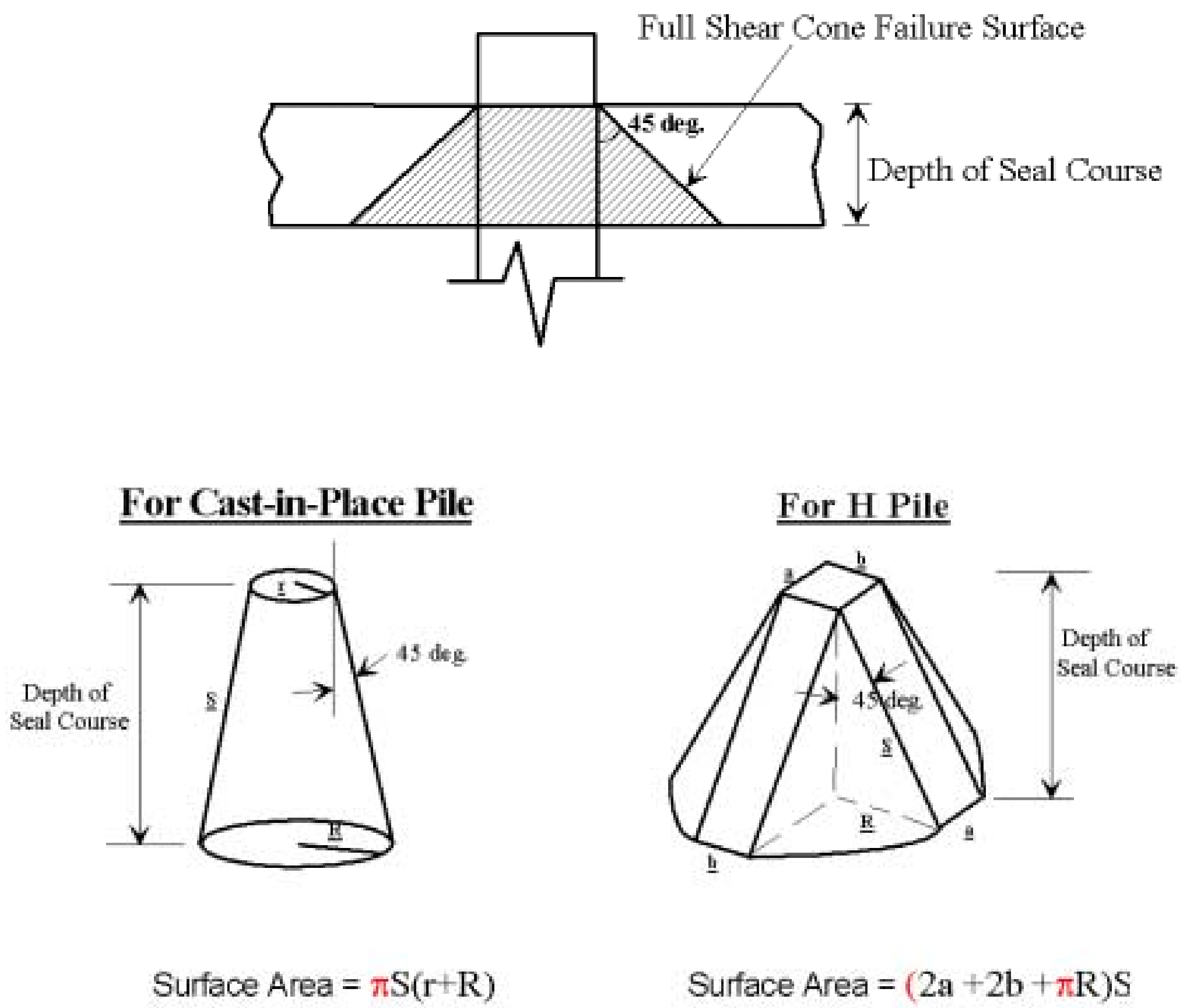
AASHTO 4.5.6.6.1
Bridge Manual 3.74 page 1.1.1

(3) Maximum pullout of 20 kips for C.I.P. piles. (This is based on 33% of the 14" CIP pile's maximum frictional pile load.)

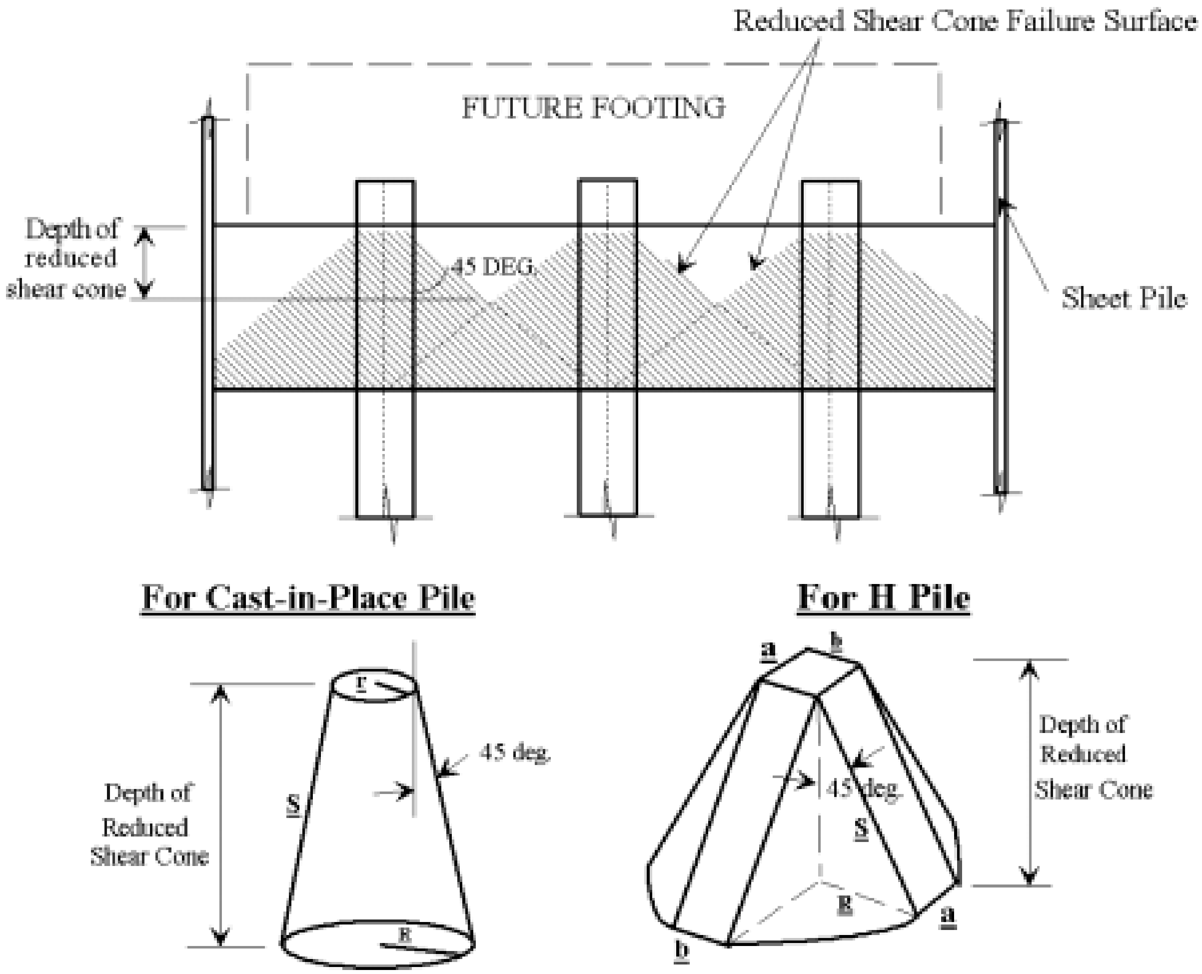
AASHTO 4.5.7.3

(4) Maximum pullout of $0.25P_y$ for steel pile. P_y is the yield axial load of steel pile.
For C.I.P. pile, check items (1), (2), and (3) only.
For steel pile, check items (1), (2), and (4) only.

<u>Shear Cone Failure Surface</u>



<u>Figure 1 – Full Shear Cone Failure Surface</u>



Reduced
Surface Area = $\pi S(r+R)$

Reduced
Surface Area = $(2a+2b+\pi R)S$

Figure 2 – Reduced Shear Cone Failure Surface

The reduced surface area defined in Figure 2 is a conservative value compared to the actual reduced surface area. The geometry of the actual surface area is time consuming and complicated to compute, so a standard shape was chosen to ensure efficient use of the designer's time.

Example 1 - Seal Course

Problem

Check if Seal Course design is adequate.

Given

- Concrete Strength f'c= 3000 psi
- Pile spacing = 3'-0"
- 12 - HP 10 x 42 piles
- Area of an individual HP 10 x 42 Pile = A = 12.4 in^2
- Yield strength of steel F_y = 36 ksi
- Maximum axial load $P_y = F_y (A)$ = 36 ksi (12.4 in^2) = 446.4 kips
- Hydrostatic head h = 19'
- Seal Course = 12' x 15' x 3'
- Pile embedment below seal course = 15'

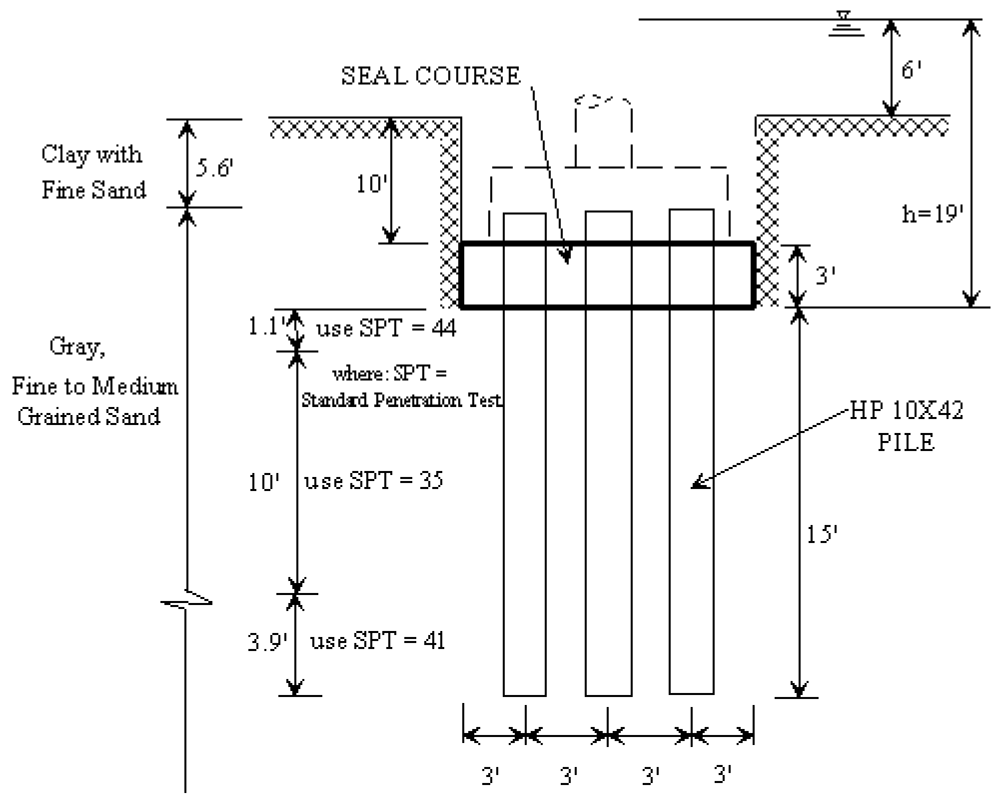


Figure 3 – Seal Course Elevation

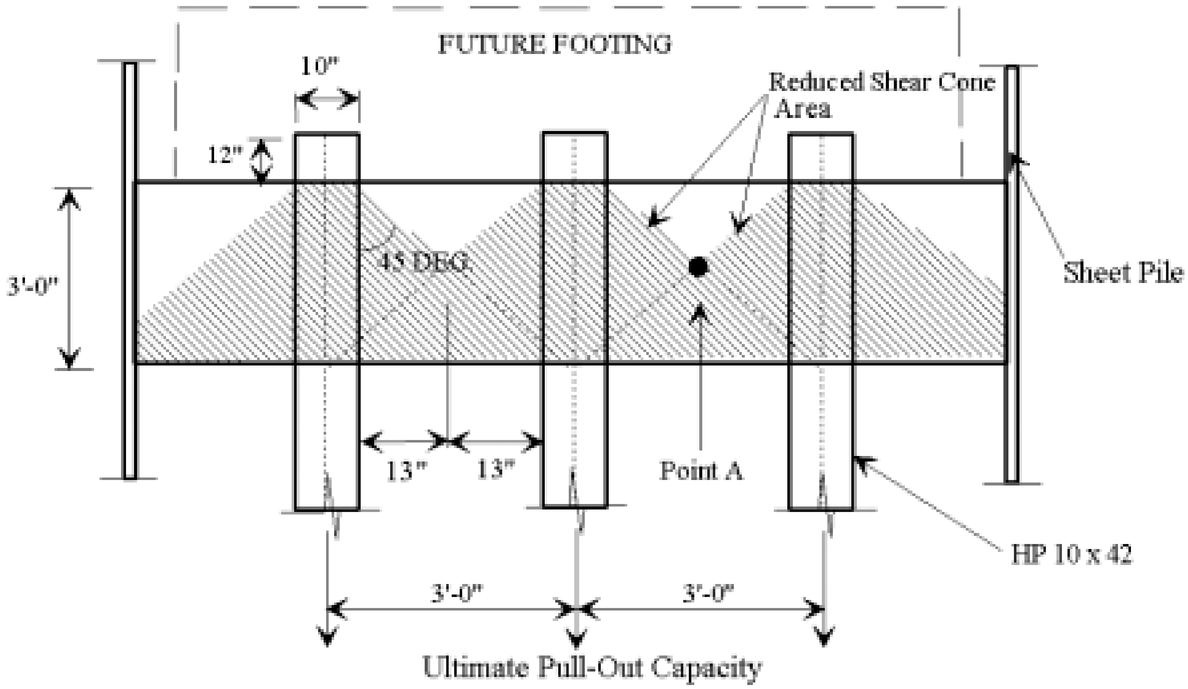


Figure 4 – Typical Shear Cone Failure Area

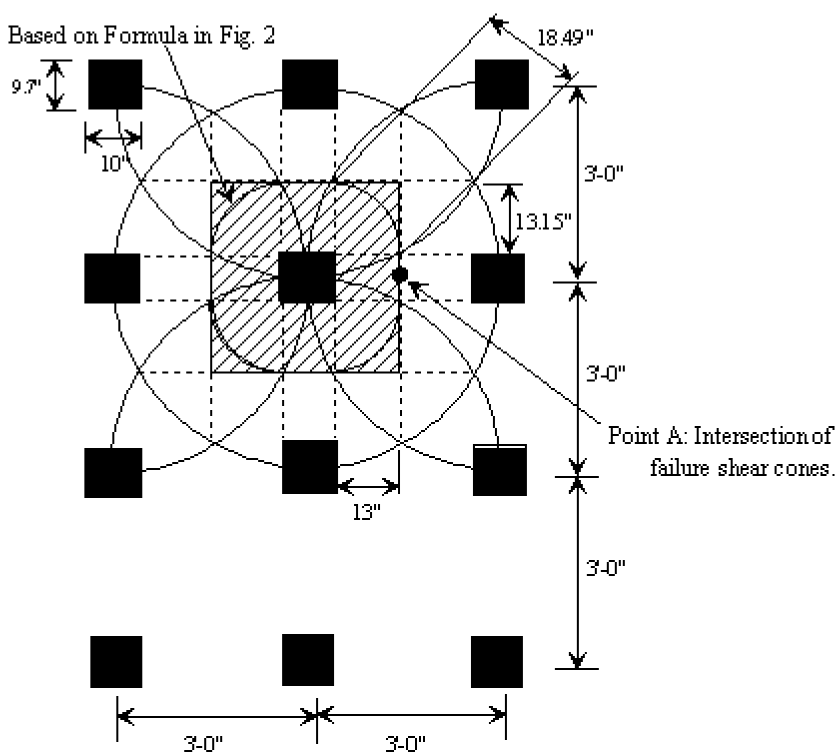


Figure 5 – Shear Failure Area for Individual Pile

Solution

ACTUAL UPLIFT FORCE:

Uplift force of water = 12'(15')(19')(0.0624 kips/ft^3)	=	214 kips
Weight of seal course = 12'(15')(3')(0.15 kips/ft^3)	=	-81 kips
Friction of sheet pile = (15'+12')(2)(3')(144)(0.002kips/in^2)	=	-47 kips
Net uplift of piles	=	86 kips

Actual uplift per pile = 86 kips/12 piles = 7.17 kips/pile

ALLOWABLE UPLIFT FORCE:

Use the minimum of ....

(1) Allowable friction capacity of pile:

Using "SPILE" program, the ultimate friction capacity of pile is 53.97 kips.
Allowable friction capacity = 53.97 kips/3.5 = 15.42 kips

(2) Allowable pullout capacity of pile due to shear cone failure:

(Reference Figures 4 & 5)

- Total Reduced Shear Cone Area for H-pile
  = $(2a + 2b + \pi R)S$
  = (2 x 9.7" + 2 x 10" + 3.1416 x 13") x 18.38"
  = 1474 in^2

- Ultimate Shear Strength
  = $2\sqrt{f'c}$
  = $2\sqrt{3000}$
  = 109.5 psi

- Total Pull-Out Capacity
  = (Total Shear Cone Area) x (Ultimate Shear Strength)
  = 1474 in^2 x 0.1095 ksi
  = 161.4 kips $\leq P_y$ = 446.4 kips

- Allowable Pull-Out Capacity
  = Total Pull-Out Capacity / Factor of Safety
  = 161.4 kips/3
  = 53.8 kips < $0.25P_y$ = 0.25 x 446.4 kips = 111.6 kips, O.K.

From (1) & (2):
Allowable uplift force = 15.42 kips
Actual uplift force = 7.17 kips

15.42 kips > 7.17 kips O.K.

Try seal course depth = 2'-6"

ACTUAL UPLIFT FORCE:

Uplift force of water = 12'(15')(18.5')(0.0624 kips/ft^3)	=	207.8 kips
Weight of seal course = 12'(15')(2.5')(0.15 kips/ft^3)	=	-67.5 kips
Friction of sheet pile = (15'+12')(2)(2.5')(144)(0.002 kips/in^2)	=	-38.9 kips
Net uplift of piles	=	101.4 kips

Actual uplift per pile = 101.4 kips/12 piles = 8.45 kips/pile

ALLOWABLE UPLIFT FORCE:

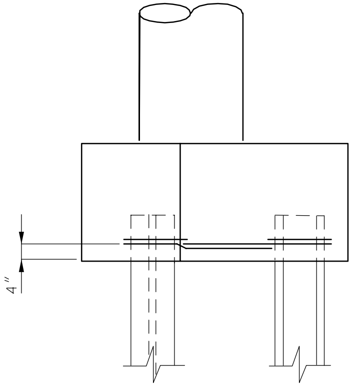
Use the minimum of ....

(1) Allowable friction capacity of pile:
Using "SPILE" program, the ultimate friction capacity of pile is 54.37 kips.
Allowable friction capacity 54.37 kips/3.5 = 15.53 kips

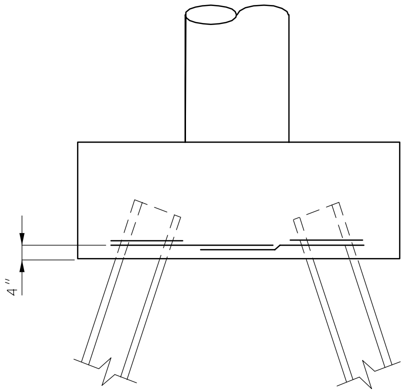
(2) Allowable pullout capacity of pile due to shear cone failure = 53.8 kips

From (1) & (2):
Allowable uplift force = 15.53 kips
Actual uplift force = 8.45 kips
15.53 kips > 8.45 kips
(Thus, using a depth of 2'-6" is an economical design.)

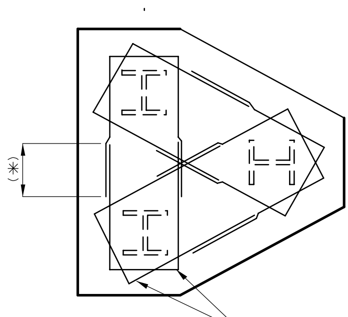
REINFORCEMENT
UNREINFORCED FOOTING - USE ONLY IN SEISMIC PERFORMANCE CATEGORY A

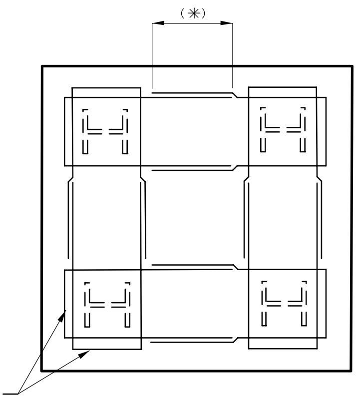


ELEVATION
(3 PILE FOOTING)



ELEVATION
(4 PILE FOOTING)





#6 Hairpin Bars (Typ.)

PLAN
(3 PILE FOOTING)

PLAN
(4 PILE FOOTING)

Notes:

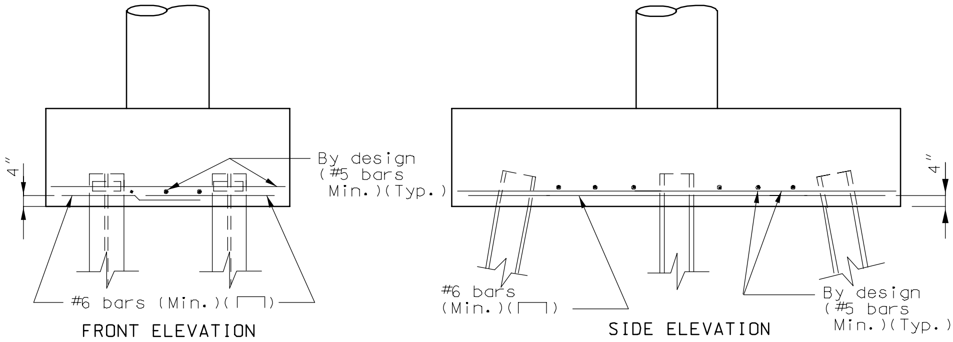
(✳) See lap splice class C (Other than top bars) - In Manual Section 2.4.

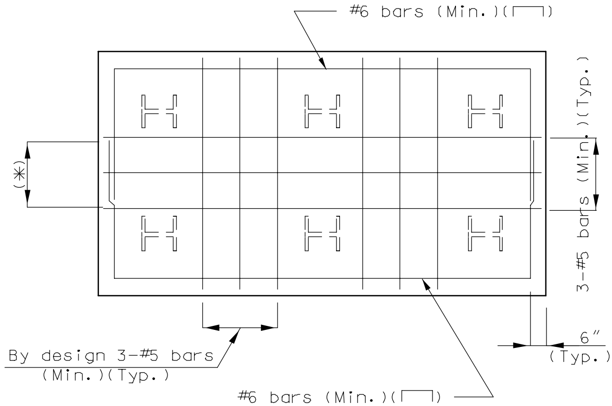
Reiforcement not required by design. Hairpins are sufficient for renforcing requirements.

The minimum percentage of reinforcement, "P" , is not required to be met, unless scour is anticipated.

Use for all types of piling, except timber.

REINFORCEMENT(CONT.) Pile Footing
REINFORCED FOOTING - SEISMIC PERFORMANCE CATEGORY A





PLAN

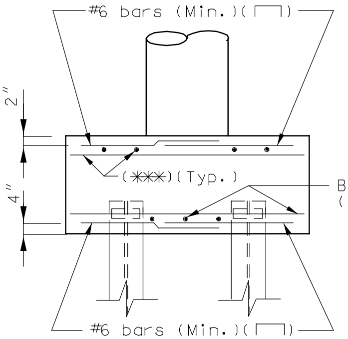
Notes:

(*) See lap splice class C (Other than top bars) - In manual Section 2.4.

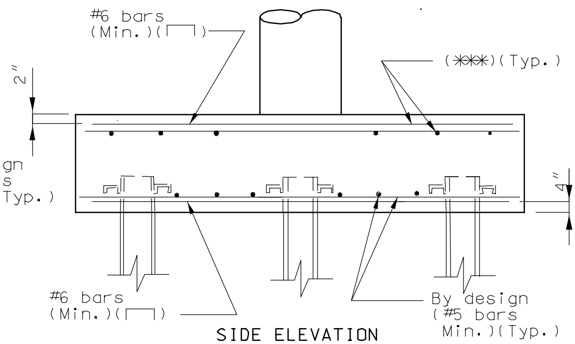
The maximum size of stress steel allowed is #8 bars.

REINFORCEMENT
Pile Footing

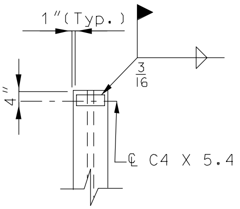
REINFORCED FOOTING - SEISMIC PERFORMANCE CATEGORIES B, C & D



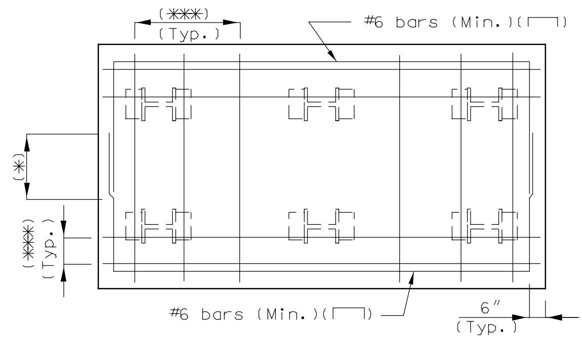
FRONT ELEVATION



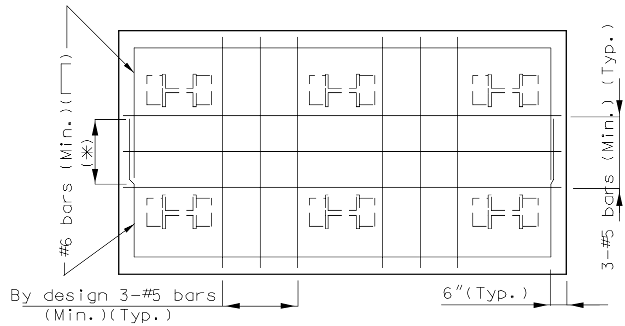
SIDE ELEVATION



TYPICAL DETAIL
PILE CHANNEL
SHEAR CONNECTOR



PLAN SHOWING TOP REINFORCEMENT



PLAN SHOWING BOTTOM REINFORCEMENT

Notes:

For reinforcement in bottom of the footing, see lap splice Class C (Other than top bars) - In Manual Section 2.4.

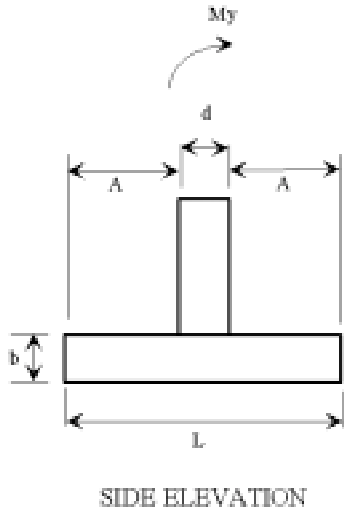
(✱) For reinforcement in top of the footing, see lap splice class C (Top bars) - In Manual Section 2.4.

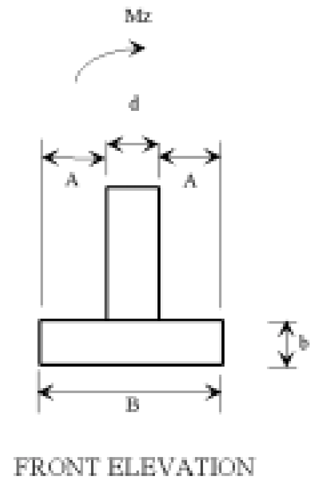
(✱✱) Place the top reinforcement uniformly outside the column.

(✱✱✱) Use same area of steel in the top of the footing as is required for the bottom,
The maximum size of stress steel allowed is #8 bars.

Unreinforced footings shall not be used in seismic performance categories B, C & D.

Design and Dimensions





d - column diameter
L - footing length
b - footing depth
B - footing width
A - edge distance from column

Dimensional Requirements

L - Minimum of 1/6 x distance from top of beam to bottom of footing (3" increments);

B - Minimum footing width is column diameter + 2A, (3" increments);

A - Minimum of 12";

b - Minimum of 30" or column diameter, Maximum of 72" at 3" increments; (for column diameters 48" and greater use a 48" minimum footing depth.)

Design and Dimensions (Cont.)

Size

The size of footing shall be determined by computing the location of the resultant force and by calculating the bearing pressure.

Long, narrow footings are to be avoided, especially on foundation material of low capacity. In general, the length to width ratio should not exceed 2.0, except on structures where the ratio of the longitudinal to transverse loads or some other consideration makes the use of such a ratio limit impractical.

Location of Resultant Force

The location of the resultant force shall be determined by the following equations.

The Middle 1/3 is defined as: $\frac{e_L}{L} + \frac{e_B}{B} \le \frac{1}{6}$

The Middle 1/2 is defined as: $\frac{e_L}{L} \le \frac{1}{4}$ and $\frac{e_B}{B} \le \frac{1}{4}$

The Middle 2/3 is defined as : $\frac{e_L}{L} \le \frac{1}{3}$ and $\frac{e_B}{B} \le \frac{1}{3}$

The following table specifies requirements for the location of the resultant force.

Soil Type	Resultant Location Group I - VI	Resultant Location Earthquake Loads Categories B, C and D
Clay, clay and boulders, cemented gravel, soft shale with allowable bearing values less than 6 tons, etc.	middle 1/3	middle 1/2
Rock, hard shale with allowable bearing values of 6 tons or more.	middle 1/2	middle 2/3

Design and Dimensions (Cont.)

Bearing Pressure

The bearing pressure for Group I thru VI loads shall be calculated using service loads and the allowable overstress reduction factors as specified in AASHTO Table 3.21.1A. The calculated bearing pressure shall be less than the allowable pressure given on the Design Layout.

The bearing pressure for Earthquake Loads in Categories B, C, and D shall be calculated from loads specified in AASHTO Division I-A Seismic Design, Sections 6.2.2, 7.2.1, and 7.2.2. The seismic design moment shall be the elastic seismic moment (EQ) divided by the modified response modification factor R'. The modified seismic moment shall then be combined independently with moments from other loads:

AASHTO Division I-A Table 3

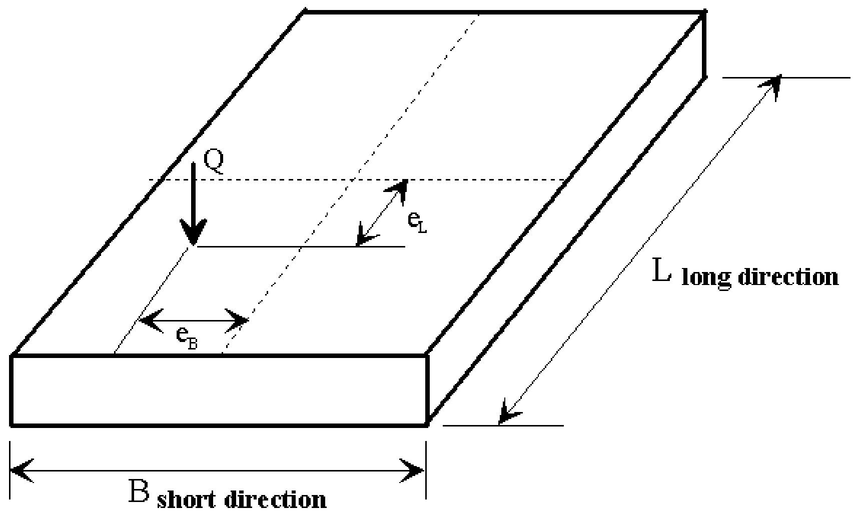
Group Load = 1.0(D + B + SF + E + EQ/R')

where D = dead load
B = buoyancy
SF = stream flow pressure
EQ = elastic seismic moment
E = earth pressure
R' = R/2 for category B
= 1 for categories C and D

R = Response Modification Factor
= 5 for multi-column bent
= 3 for single-column bent

The calculated bearing pressure shall be less than the ultimate capacity of the foundation soil. The ultimate capacity of the foundation soil can be conservatively estimated as 2.0 times the allowable bearing pressure given on the Design Layout. The analysis method of calculating bearing pressures is outlined in the following information.

See AASHTO 4.4.2 for explanation of notations.



Sketch of Dimensions for Footings Subjected to Eccentric Loading

For $e_L < L/6$

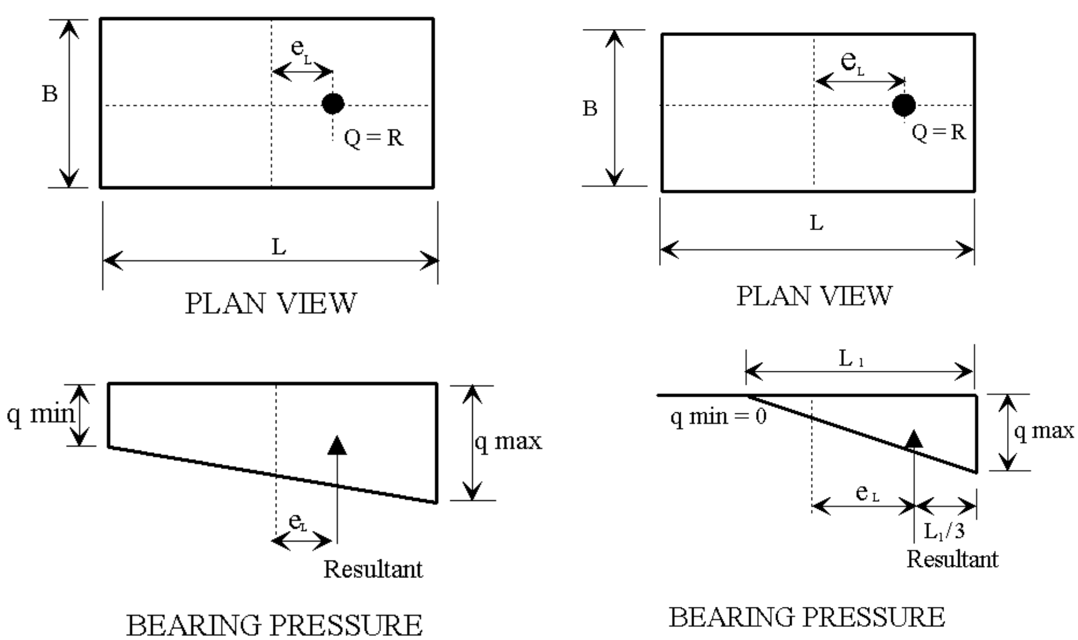
$$q_{\max} = \frac{Q(1+\frac{6e_L}{L})}{BL}$$

$$q_{\min} = \frac{Q(1-\frac{6e_L}{L})}{BL}$$

For $L/6 < e_L < L/2$

$$q_{\max} = \frac{2Q}{3B(L/2-e_L)}$$

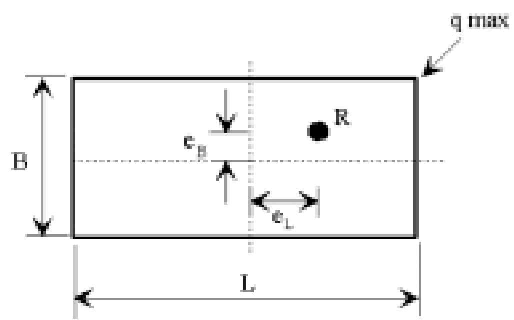
$$q_{\min} = 0, \quad L_1 = 3(L/2-e_L)$$



Bearing Pressure for Footing Loaded Eccentrically About One Axis

Variables determined from AASHTO Figure 4.4.7.1.1.1C

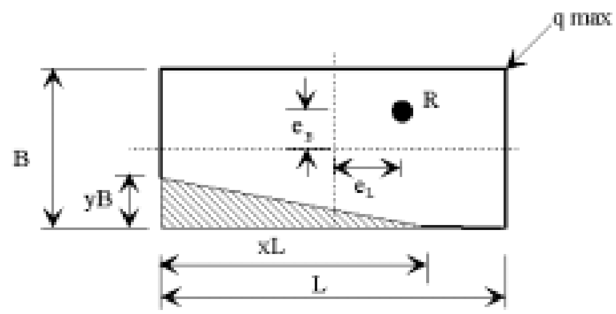
CASE 1



$$q\ max = \frac{R}{BL}\left(1 + \frac{6e_L}{L} + \frac{6e_B}{B}\right)$$

CASE 1 Plan View

CASE 2

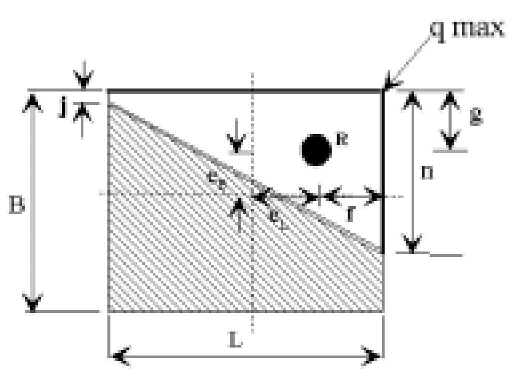


k, x, and y from AASHTO chart

$$q\ max = \frac{KR}{BL}$$

CASE 2 Plan View

CASE 3



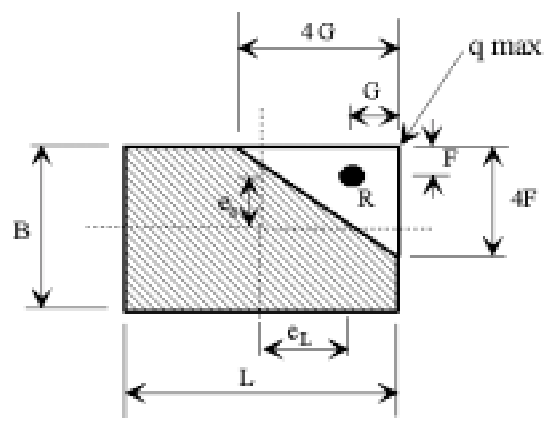
$r = j/n$ $s = 1 + r + r^2$

$g = \frac{n(1 + r\,s)}{4s}$ $f = \frac{L(3s - r - 2)}{4s}$

$$q\ max = \frac{6R}{L\,n\,s}$$

CASE 3 Plan View

CASE 4



$$q\ max = \frac{3R}{8FG}$$

CASE 4 Plan View

<u>Bearing Pressure for Footing</u>
<u>Loaded Eccentrically About Two Axes</u>

Loading Cases

AASHTO - ARTICLE 3.22

Loads for Groups I thru VI shall be calculated for all bridges.

Earthquake loads shall be calculated when the bridge is in Seismic Zones B, C, and D.

Loads for other group loadings shall be used on a case by case basis.

Reinforcement

The footing is to be designed so that the shear strength of the concrete is adequate to handle the shear stress without the additional help of reinforcement. If the shear stress is too great, the footing depth should be increased.

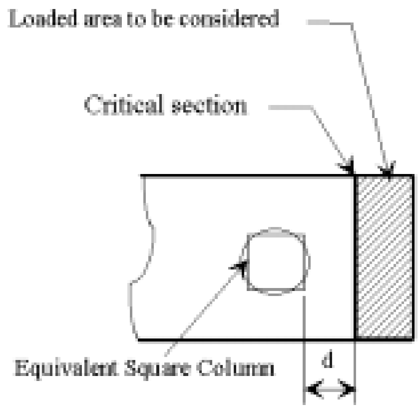
Shear

The shear capacity of the footings in the vicinity of concentrated loads shall be governed by the more severe of the following two conditions.

AASHTO Articles 8.15.5 or 8.16.6

AASHTO 8.15.5.6

Critical section at "d" distance from face of column:



Load Factor

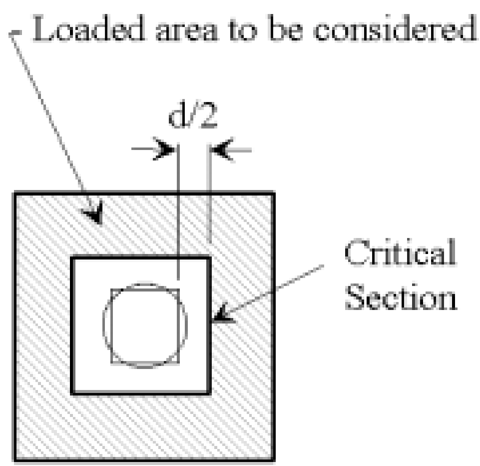
$$V_n = V_u /(\phi \ bd)$$

$$V_c = 2\sqrt{f'c}$$

b = footing width

AASHTO 8.16.6.6

Critical section at "d/2" distance from the column:



Load Factor

$$V_n = V_u /(\phi \ b_0 d)$$

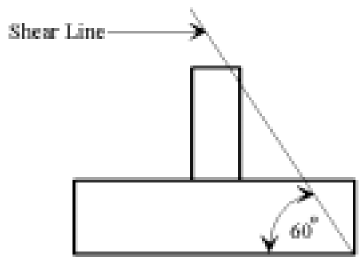
$$V_c = 4\sqrt{f'c}$$

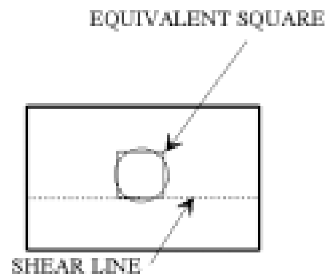
b_0 = 4(d + Equivalent square column width)
If shear stress is excessive, increase footing depth.

Bending

If the shear line is within the projected equivalent square column, the footing may be considered satisfactory for all conditions. (minimum reinforcement required)

If the shear line is outside of the projected column, the footing must be analyzed and reinforced for bending and checked for shear stress.





The critical section for bending shall be taken at the face of the equivalent square column. The equivalent square column is the theoretical square column which has a cross sectional area equal to the round section of the actual column and placed concentrically.

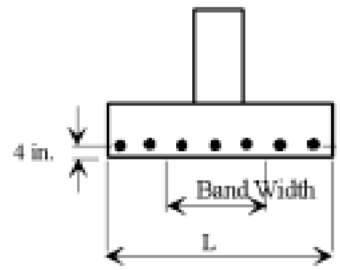
Reinforcement in Bottom of Footing

The bearing pressure used to design bending reinforcement for Group I thru VI loads shall be calculated using Load Factor Loads.

The bearing pressure used to design bending reinforcement for Earthquake Loads in Categories B, C, and D shall be calculated from the same loads as specified in AASHTO Division 1-A Seismic Design for ultimate bearing pressure.

The bottom reinforcement shall be designed using ultimate strength design.

Distribution of Reinforcement



L = Footing Length

B = Footing Width

Reinforcement shall be distributed uniformly across the entire width of footing in the long direction. In the short direction, the portion of the total reinforcement given by AASHTO Equation 4.4.11.2.2-1 shall be distributed uniformly over a band width equal to the length of the short side of the footing, B.

Band Width Reinforcement = 2(total reinforcement in short direction)/(β+1)
β = the ratio of footing length to width = L/B

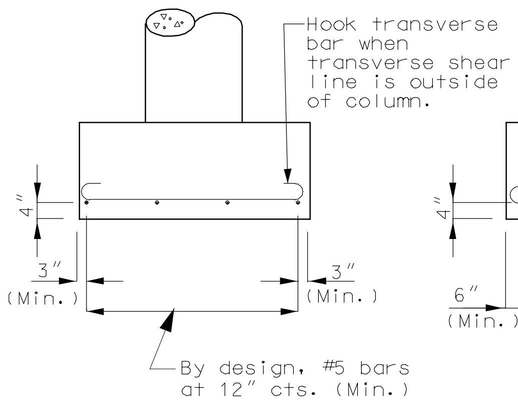
The remainder of the reinforcement required in the short direction shall be distributed uniformly outside the center band width of footing.

Reinforcement in Top of Footing

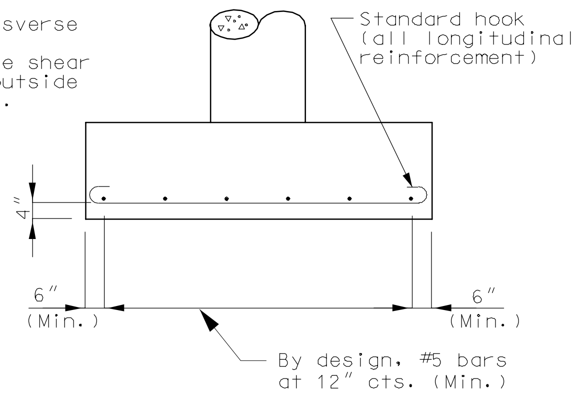
Reinforcement in the top of the footing shall be provided for Seismic Performance Categories B, C, and D. This reinforcement shall be the equivalent area as the bottom steel in both directions. The top steel shall be placed uniformly outside the column.

REINFORCEMENT DETAILS - SEISMIC PERFORMANCE CATEGORY A

Spread Footings

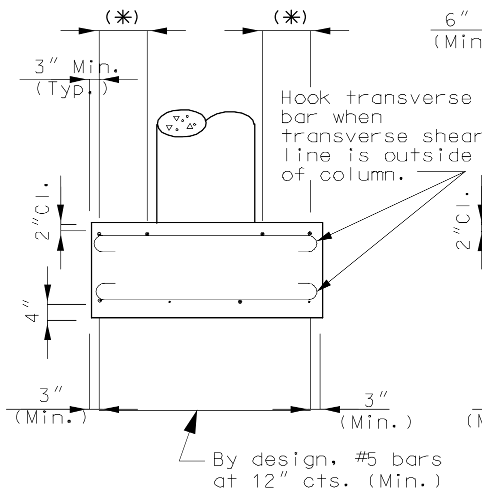


FRONT ELEVATION

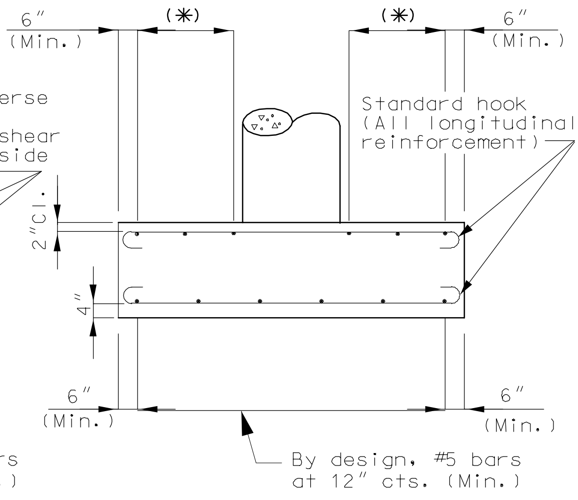


SIDE ELEVATION

REINFORCEMENT DETAILS - SEISMIC PERFORMANCE CATEGORIES B, C & D



FRONT ELEVATION



SIDE ELEVATION

(✳) Use same area of steel in the top of the footing as is required for the bottom.